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16. ABSTRACT

An indirect source, in this guideline, is any facility attracting mobile source (i.e., motor vehicle) activity with carbon monoxide, CO, emissions. These guidelines provide a comprehensive, manual methodology to assess both the one- and eight-hourly CO impact of indirect sources. This methodology encompasses a three-part procedure: First, the physical characteristics of the roadway/parking area network and the projected traffic demand volume are used to determine various aspects of the traffic flow (e.g., delay, queue length, parking area running time). Second, these traffic features, together with other ambient parameters (e.g., year, temperature, geography, hot/cold start ratio), are used to determine accompanying modal CO emission rates. Third, these emissions are input to an atmospheric dispersion analysis that considers variations in source type (i.e., infinite line, finite line, and area), wind speed and direction, stability, road/receptor orientation, and terrain roughness. The evaluation procedure is capsulized using a series of annotated worksheets, graphs, and tables. Supplemental information is provided in appendices that should eliminate the need for additional references in most cases.

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GUIDELINES FOR AIR QUALITY MAINTENANCE PLANNING AND ANALYSIS
VOLUME 9 (REVISED): EVALUATING INDIRECT SOURCES

U.S. ENVIRONMENTAL PROTECTION AGENCY
Office of Air, Noise, and Radiation
Office of Air Quality Planning and Standards
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The guideline series of reports is being issued by the Office of Air Quality Planning and Standards (OAQPS) to provide information to state and local air pollution control agencies; for example, to provide guidance on the acquisition and processing of air quality data and on the planning and analysis requisite for the maintenance of air quality. Reports published in this series will be available - as supplies permit - from the Library Services Office (MD-35), U.S. Environmental Protection Agency, Research Triangle Park, North Carolina 27711; or, for a nominal fee, from the National Technical Information Service, 5285 Port Royal Road, Springfield, Virginia 22161.

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FOREWORD

This document is the ninth in a series comprising Guidelines for Air Quality Maintenance Planning and Analysis. The intent of the series is to provide State and local agencies with information and guidance for the preparation of Air Quality Maintenance Plans. The volumes in this series are:

Volume 1: Designation of Air Quality Maintenance Areas
Volume 2: Plan Preparation
Volume 3: Control Strategies
Volume 4: Land Use and Transportation Consideration
Volume 5: Case Studies in Plan Development
Volume 6: Overview of Air Quality Maintenance Area Analysis
Volume 7: Projecting County Emissions
Volume 8: Computer-Assisted Area Source Emissions Gridding Procedure
Volume 9: Evaluating Indirect Sources
Volume 10: Reviewing New Stationary Sources
Volume 11: Air Quality Monitoring and Data Analysis
Volume 12: Applying Atmospheric Simulation Models to Air Quality Maintenance Areas
Volume 13: Allocating Projected Emissions to Sub-County Areas

Additional volumes may be issued.

These guidelines are intended to provide a detailed manual technique for evaluating proposed indirect sources. This document supersedes "Volume 9: Evaluating Indirect Sources," EPA-450/4-75-001, OAQPS No. 1.2-028, January, 1975. The information in this volume is being published at this time primarily to assist those persons having responsibility for estimating roadside carbon monoxide (CO) concentrations as the result of requirements imposed by environmental impact statement analysis and review and analysis for air quality maintenance plans.

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Key Terms

The key terms used in this document are defined as follows:

Key Word	Definition
Average highway speed	The weighted average of the design speeds within a highway section, when each subsection within the section is considered to have an individual design speed.
Area source	An extended region (e.g., parking lot) in which pollutant emissions are reasonably uniformly distributed.
Background concentration	The concentration of pollutants in the air at a receptor that are the result of emissions outside the local vicinity; the concentration at the upwind edge of a local source. (mg m^{-3} or ppm)
Cold starts* (%)	The percentage of vehicles in the projected 1- and 8-hour demand volumes, which were started cold and have been running less than 8.5 minutes.
Cruise speed	The highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions without at any time exceeding the safe speed as determined by the design speed on a section-by-section basis.
Dispersion	The amount of dilution of an air contaminant resulting from the combined effects of transport (wind speed) and diffusion (turbulent mixing).
Left turns (%)	The percentage of vehicles turning left at the intersection.

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*In the FTP, the emissions corresponding to the cold transient cycle.

Key Word	Definition
Line source	A configuration of pollutant emissions that can be approximated by a single line. Thus, CO emitted by vehicles moving in a traffic lane can be reasonably well approximated as having originated from a single line in the center of the lane.
Local concentration	The concentration at a receptor of a particular air pollutant (e.g., CO) that is the result of emission from a local source in proximity to the receptor. The sum of the local and background concentrations is the total receptor concentration. ($mg\ m^{-3}$ or ppm)
Metropolitan population	The number of persons residing in the greater metropolitan area.
Parking lot stall	A lane for parking vehicles. A stall may store more than one vehicle, as when several vehicles are parked end to end with only the first and last vehicle having access to a moving lane of traffic.
Receptor	A specialized location where pollutant concentrations are either measured or computed. Unless otherwise specified, receptors are assumed to have a height above ground of about 1.8 m--the height of the typical breathing zone.
Right turns (%)	The percentage of vehicles turning right at the intersection.
Source	The origin of pollutant emissions; motor vehicle sources are usually represented as either line sources or area sources.

Key Word	Definition
Total concentration	The actual concentration expected or measured at a receptor; the sum of the local and backgroupd concentration components (mg m^{-3} or ppm).
Trucks and buses (%)	The percentage of the vechicles in the traffic volume that are heavy duty trucks and buses.

ABBREVIATIONS AND SYMBOLS

Abbreviations

deg	degree	mg	milligram
ft	feet	mph	miles per hour
g	grams	ppm	parts per million
hr	hour	sec,s	second
m	meter	veh	vehicle

Symbols	Definition
A	The area used for parking and access at a facility. (m^2)
Brt (or BRT)	Base running time: The sum of the base approach, base entrance, base movement-in, base stop, base start, base movement-out, base exit, and base departure times. (sec)
C	Capacity: The number of vehicles that can pass along a given section of roadway in an hour if the flow is uninterrupted. (veh hr^{-1})
Cf	A correction factor applied to parking lot running time when the lot is more than 80% full.
Cs	Capacity service volume: A service volume is the maximum number of vehicles that can pass over a given section of roadway during a time period while operating conditions are maintained corresponding to a specified level of service. Capacity occurs at level of service E. It represents the most vehicles that an approach can accommodate during 1 hour of green time. (veh hr^{-1})

Symbols	Definition
C_T	Total emission correction factor: a correcting factor that converts emissions to varying % of cold starts and hot starts, ambient temperatures, speeds, and calendar years.
Cy	Cycle length: The number of seconds for one complete sequence of signal indicators (sec).
D	Average stopped delay: The average delay experienced by all vehicles that stop on one approach to a signalized intersection (sec).
Ea	The excess emissions above the free flow emissions produced by a vehicle when accelerating to cruise speed at 2.5 mph/s (g veh^{-1} m^{-1})
Ed	The excess emissions above the free flow emissions produced by a vehicle when decelerating from cruise speed at 2.5 mph/s (g veh^{-1} m^{-1})
Ef	The emissions at free flow produced by a vehicle at cruise speed (g veh^{-1} m^{-1})
F	CO emission rate for slow moving vehicles used in calculating areawide emissions at a parking lot (g sec^{-1} veh^{-1})
Fet	Facility emptying time: The time it takes for all spectators to reach their vehicles after the end of an event (sec).
Fs	Emission rate from a queue of stopped or slowly moving vehicles ($F_s = Q_s \cdot L_q$) (g sec^{-1}).
G	The interval of time of the green signal indicators allocated to any traffic movement or combinations of traffic movements (sec).

Symbols	Definition
H	Mixing height: Height above ground of the atmospheric layer within which surface emissions can be mixed; usually determined by the base of the lowest elevated inversion (m).
i	A numerical designation used as a subscript to identify intersection legs, sections of roadway, or gates to a parking facility.
j	Signal phase designation: A subscript used to denote each green signal indication that allows one or more movements of traffic through an intersection.
Lad	The length of roadway required to decelerate to a stop from cruise speed, S, or to accelerate back to cruise speed at the rate of 2.5 mph/s (m).
Le	The length of roadway in which excess emissions will be considered to occur (m).
Lf	Length of a road segment of free flow emissions (m).
Lq	The maximum length of the vehicle queue (m).
M	The number of traffic lanes comprising the approach.
N	The number of vehicles that stop during a signal cycle (i.e., during phase green and phase red) (see equation 4 Chapter III).
Np	The number of green signal phases at a signalized intersection.
P	The proportion of vehicles that must stop at least once at an intersection.
Pc	The number of vehicles that can park in a parking lot.
Plet (or PLET)	Parking lot emptying time: The time it takes all parked vehicles to exit an event oriented facility parking lot (seconds).

Symbols	Definition
Po	The number of vehicles occupying stalls in a parking lot.
Q	Emission rate per unit length is a measure of the amount of air pollutant emitted on 1 meter of roadway each second (g m^{-1} sec^{-1}).
Qa	The emission rate averaged over an area (g m^{-2} sec^{-1}).
Qe	The excess emission rate that exists over a length of roadway, Le (g m^{-1} sec^{-1}).
Qf	The emission rate emitted by a vehicle at cruise speed under free flow conditions (g m^{-1} sec^{-1}).
Qs	The excess emission rate for idling vehicles (g m^{-1} sec^{-1}).
Rmo	The time spent waiting to move out of a parking lot (sec).
Rp	The time required to move to an auxiliary parking lot, if the main lot is full (sec).
Rq	Average time spent waiting (1) to enter or exit a parking lot through one of the gates or (2) to continue through an intersection (sec/veh).
Rt	Typical vehicle running time when making one trip into or out of the area of interest (sec).
S	The average speed of a vehicle (mph).
SC	Stability class: A measure of the hydrostatic equilibrium of the atmosphere. Stability can be classified into groups denoted by letters of the alphabet. Class D refers to neutral conditions, A-C to unstable, and E,F to stable. Pollutant dispersion is increasingly greater as the stability decreases (i.e., from F toward A).

Symbols	Definition
T	The running time for a typical vehicle trip during a 1-hour period of interest (sec).
U	Wind speed (m s^{-1}).
V_i (V)	Volume (demand): The number (or projected number) of vehicles using an approach i during a specified time period (veh hr^{-1}).
Va	The number of vehicle arrivals at a parking facility that will park in 1 hour. This figure should not include mass transit or other passenger-delivering vehicles that do not park (veh hr^{-1}).
W	Intersection approach width which includes any additional turn or through lanes that may influence the intersection capacity (ft).
x	The horizontal, perpendicular distance from the receptor to the line source (i.e., the center of the roadway lanes being evaluated) (m).
Yf	Yearly correction factor: Converts from base year (1975) emissions to study year emissions.
X	Concentration: The mass of a particular pollutant contained within a unit volume of air (g m^{-3}), or the volume occupied by a particular pollutant within a unit volume of air (ppm).

Symbols	Definition
$\chi U/Q$	Normalized concentration: The product of the concentration and wind speed, divided by the emission rate. The normalized concentration is a measure of the magnitude of the atmospheric dispersion of the contaminant due to turbulent mixing. For a line source, the units are (m^{-1}), while for an area source the value is nondimensional.
σ_z	Dispersion coefficient: The standard deviation in the vertical of the plume concentration distribution (m).
σ_{z_o}	An initial vertical standard deviation (m) of a source plume used to account for any initial mixing at the source.
θ	Wind/road angle which is formed at the intersection of the wind vector and the roadway axis (degrees).

I. INTRODUCTION

A. Purpose of the Guidelines

These guidelines are designed to evaluate the impact of an indirect source on the air quality. An indirect source is defined as a facility, building, structure, or installation, attracting mobile activity with carbon monoxide (CO) emissions. Examples of indirect sources include, but are not limited to:

- Highways and roads
- Parking lots and garages
- Airports
- Retail, commercial, industrial, educational, amusement, sport, and entertainment facilities
- Office and government buildings
- Apartment, condominium, and housing projects

The explicit purposes of this document are:

- To provide means for estimating whether an indirect source may exceed the National Ambient Air Quality Standards (NAAQS) for carbon monoxide (CO)
- To provide a means of evaluating the efficacy of good traffic engineering practice in meeting standards for CO.

The procedures have been developed to be comprehensive in scope and realistic in detail. To aid in its application, the evaluation procedure is outlined in Chapter III and is summarized in a series of worksheets and flow charts; tables and illustrations are also presented to facilitate user application.

B. Rationale and Scope of the Evaluation Procedure

To evaluate the air quality impact of an indirect source, the incremental air pollution induced by the facility must be added to ambient levels at the site and the total compared to ambient air quality standards for CO. Thus, the sum of two components is considered to be the projected CO concentration at the site:

- Background concentrations at the upwind edge of the site
- Local contribution resulting from CO emissions at and near the proposed facility.

Furthermore, the latter component consists of two elements:

- CO emissions resulting from motor vehicle traffic induced by the facility
- CO emissions from other (or "through") traffic.

The dominance of the various combinations of "components" and "elements" often depends on the hour, day, season, and length of time under consideration. Specifically, the indirect source evaluation methodology must consider both the worst-case one-hourly and eight-hourly periods because it is a preliminary technique and the results are based on a very limited data base. As a general (although not absolute) rule, local CO contributions dominate the total CO concentration for the worst one-hourly cases. On the other hand, the worst eight-hourly cases are frequently dominated by high background contributions of CO. The indirect source guidelines seek to assess the eight-hourly situation as realistically as practicable by treating the eight-hour CO projection as the average of eight one-hourly analyses. This approach allows the user to retain actual worst-case hour-to-hour variations of traffic demand volume, wind speed and direction, stability, and background.

The local CO contribution due to most indirect sources can be considered as the summation of contributions from one or more of three types of sources:

- Extended line sources
- Finite line sources
- Area sources

This categorization defines the structure of the guidelines. At a receptor location the contribution of the total CO concentration from each source type is calculated in a three-step process. In the first step, (1) the network description and traffic demand volume are used to estimate the traffic flow characteristics. Emissions are computed in step (2). For an extended roadway, emissions are computed on the basis of vehicle speed and volume, and are assumed to be uniform along the roadway. At an intersection, emissions are treated as the sum of two parts. One part is the emissions produced when all vehicles are nonstopping vehicles; the second part is the excess emissions emitted over a finite length by stopping vehicles. For an area source (e.g., parking lot), emissions are assumed to be uniform over the area and are derived from total vehicle running time. Once emissions have been computed, the third step (3) in the process estimates the effect of atmospheric dispersion on actual concentrations at the specified receptor location. Graphical techniques are developed for estimating dispersion from either finite or discrete line sources and from area sources. Input variables include: stability, terrain roughness, receptor location, wind speed and direction, and size of the finite line source or area source. All dispersion estimates are at 1.8 m above ground level unless corrected using a z-correlation factor included in the worksheets. Background concentrations as a final step are considered and added to source contributions to find the total CO concentration.

Chapter II provides guidance for the selection of appropriate and reasonable receptor locations.

Chapter III discusses the three step CO concentration calculation procedure and the evaluation procedure that should be followed for each of three cases:

- When data from an historical background monitor (both with and without data from a local, two-week monitoring study) are used
- When CO data are available only from a local, two-week monitoring study
- When _no_ CO monitoring data are available.

Examples at a mid-block, intersection, and an area source receptor location are presented in Chapter IV. Although the evaluation procedure is intended to be self-contained, additional technical back-up material is given in the appendices. These deal with: methods for capacity analysis, site specific traffic guidance, a street canyon dispersion model, and a simple urban dispersion model with meteorological data inputs.

II. RECEPTOR SITING

A. Selection of Receptors

Indirect-source evaluation is the assessment of the local air quality impact of a facility. The assessment is based on projected worst-case one- and eight-hourly concentrations of carbon monoxide and how they relate to the corresponding national ambient air quality standards (NAAQS: 35 and 9 ppm, respectively). The local CO concentration is due to the summation of (1) a background or "imported" contribution and (2) locally generated emissions. The latter in turn are often due to both vehicular traffic induced by the facility being evaluated and other (through) traffic.

The locations at which concentrations are monitored or for which they are estimated are known as receptors. As a general rule, receptors should be located where:

- The maximum total projected concentration is likely to occur (not on the roadway itself)
- The general public or any significant segment thereof is likely to have access over time periods specified by NAAQS.

This usually means that receptors should be located at reasonable sites in the vicinity of these portions of the traffic network where the traffic engendered by the source combined with other traffic is likely to create the greatest amount of traffic demand or the most congestion. The recommended procedure for selecting receptor sites is through review of maps and site plans. If ambient monitoring is to be done, the evaluation procedure of Chapter III is recommended as an aid for selecting CO maxima locations. If people are not anticipated to remain at these "hot spots" for the averaging time associated with the NAAQS, then a weighted average may be more appropriate to consider. This type of averaging approximates exposure to differing CO levels over the respective time periods (see Dabberdt et al., 1974).

B. Examples of Indirect-Source Receptors

To clarify what might generally be regarded as reasonable receptor sites, a few examples are cited below. Strong emphasis is placed on the fact that these examples are generalized. In some cases a site which is ordinarily unreasonable, may, in fact, be determined to be reasonable.

1. Examples of Reasonable Receptor Sites

- All sidewalks where the general public has access on a more or less continuous basis.
- A vacant lot in which a nearby facility is planned and in whose vicinity the general public (including employees if the facility is not being intended for the prime purpose of traffic control) would have access continuously.
- Portions of a parking lot to which pedestrians have access continuously.
- The vicinity of parking lot entrances and exits, provided there is an area nearby, containing a public sidewalk, residences, or structures (e.g., an auto service center at a shopping center) to which the general public is likely to have continuous access.
- The property lines of all residences, hospitals, rest homes, schools, playgrounds, and the entrances and air intakes to all other buildings.

Generally, reasonable receptor sites should be located on:

- Occupied lot--nearest the edge within the lot to which the general public has continuous access. If this cannot be determined, the property line of the lot nearest to traffic lanes should be used.

- Vacant lot--same as for occupied lot.

- Sidewalks--sidewalks present a problem in that the general public is unlikely to occupy a relatively small portion of the walkway continuously. Nevertheless, the general public does have access to the sidewalk as a whole on a continuous basis. This suggests that it is appropriate to consider the whole sidewalk as a reasonable receptor site. This further implies that one should estimate representative CO concentrations over the sidewalk during the worst one- and eight-hourly periods.

- To estimate representative concentrations at the sidewalk, it may be necessary to average estimated concentrations along the center line of the sidewalk parallel to the roadway. In general, this longitudinal averaging should be done for each block (i.e., including two intersections and a mid-block section). This cannot be a hard and fast rule, however, since it depends on the walkway-intersection or walkway-parking lot configuration.

- Any location near breathing height (1.5 - 2.0m) to which the general public has continuous access.

2. Examples of Unreasonable Receptor Sites

- Median strips of roadways.

- Locations within the right-of-way on limited access highways.

- Within intersections or on crosswalks at intersections.

- Tunnel approaches.

- Within tollbooths.

- Portions of parking lots to which the general public is not likely to have access continuously.

3. Maximum Concentrations at Receptor Locations

When completing an indirect source evaluation, the most desirable receptor locations are those which characterize the CO problem over the broadest number of adjacent receptors and recognize the maximum impact that is likely to occur. The procedures given in Chapter III are recommended as an aid for selecting CO maxima locations, but some general guidance here is given to help the user make some preliminary decisions as to receptor locations.

The most important factors in determining maximum receptor locations at indirect sources are the following:

- Traffic congestion and/or volumes
- Proximity to facility
- Wind speed and direction (and stability)

After considering these factors the user must still be sure that the receptors are reasonable in terms of access by the general public.

For uninterrupted flow conditions the CO maxima locations are on the side of the road with the heaviest peak-hour traffic flow. The receptor should be located at a minimum perpendicular distance from the roadway, again consistent with the criteria for being a reasonable receptor site. The most practical guidance that can be given is to assume the receptor to be located at the centerline of the adjacent sidewalk or at the right-of-way limit if no sidewalk exists.

At unsignalized intersections (yield or stop-sign controlled) the receptor should be located on an approach having interrupted traffic flow, that is, where a queue may develop. If all such approaches (at unsignalized or signalized) to the intersection have an equal number of approach lanes, the receptor should be located on the approach having the highest peak volume. If the approaches have an unequal number of lanes, and the approach having the greatest number of lanes also has the highest lane volume, the receptor should be located on that approach. Lane volume can be estimated by dividing total approach volume by the number of lanes in the approach. If the approach having the highest number of lanes does not have the greatest lane volume the user must assign receptor locations to (1) the approach with the highest number of lanes, and (2) the approach with the greatest lane volume. By repeating the procedures of Chapter III a CO maximum location can be determined. In all cases mentioned above the receptor should be located at the adjacent sidewalk or at the right-of-way limit if no sidewalk exists.

For parking facilities the most reasonable guidance for maximum CO receptor siting is to place the receptor at a point downwind of a major portion of the parking lot, keeping in mind the reasonableness criteria.

III. Evaluation

A. Outline

Figure 1 outlines the general evaluation procedure for evaluating CO impact in the vicinity of indirect sources and the sequence of topics discussed in this section. The method is compartmentalized to the extent that the total CO impact is considered as the sum of the CO contribution from up to four sources:

- Local infinite line sources
- Local discrete line sources
- Local area sources
- Background.

First, the required traffic, site, and aerometric input parameters are assembled. At this time the user must identify those local line and area sources requiring assessment. Then, for each identified source, the traffic flow and resulting CO emission rates are computed. Thus, for example, an indirect source evaluation of a proposed shopping center might require the assessment of the following CO source components: (1) an adjacent freeway, (2) a major arterial with signalized intersection that serves the center's parking lot, (3) the center's parking lot, and (4) ambient background.

Once the corresponding emission "inventory" has been computed, the worst-case one-hourly CO concentration resulting from each source is calculated as a function of: (1) stability, (2) source/receptor separation, (3) wind/source orientation, (4) wind speed, and (5) terrain roughness. Lastly, the procedure provides a method for determining the corresponding

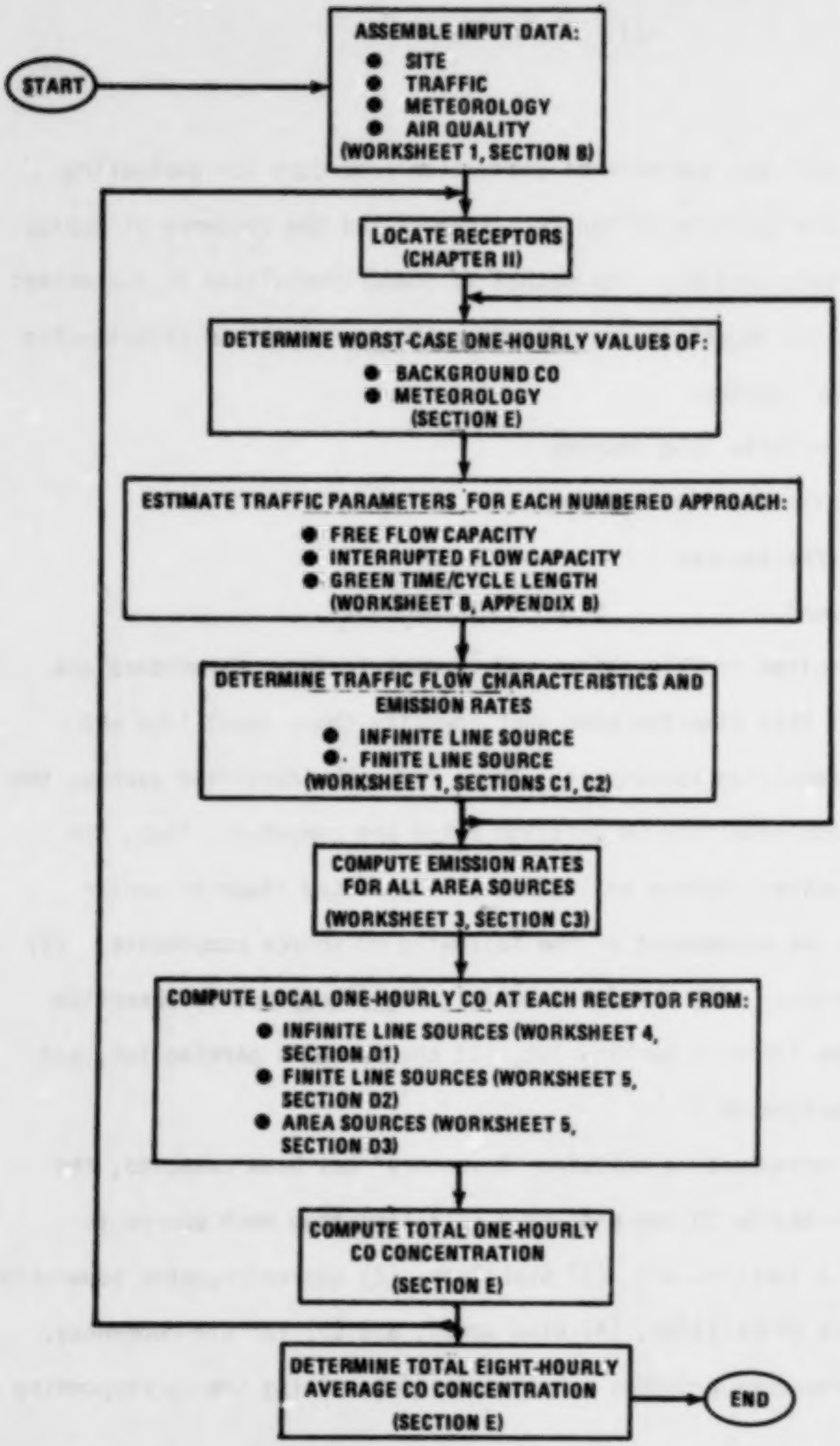
START
ASSEMBLE INPUT DATA:
● SITE
● TRAFFIC
● METEOROLOGY
● AIR QUALITY
(WORKSHEET 1, SECTION B)
LOCATE RECEPTORS
(CHAPTER II)
DETERMINE WORST-CASE ONE-HOURLY VALUES OF:
● BACKGROUND CO
● METEOROLOGY
(SECTION E)
ESTIMATE TRAFFIC PARAMETERS FOR EACH NUMBERED APPROACH:
● FREE FLOW CAPACITY
● INTERRUPTED FLOW CAPACITY
● GREEN TIME/CYCLE LENGTH
(WORKSHEET 8, APPENDIX B)
DETERMINE TRAFFIC FLOW CHARACTERISTICS AND EMISSION RATES
● INFINITE LINE SOURCE
● FINITE LINE SOURCE
(WORKSHEET 1, SECTIONS C1, C2)
COMPUTE EMISSION RATES FOR ALL AREA SOURCES
(WORKSHEET 3, SECTION C3)
COMPUTE LOCAL ONE-HOURLY CO AT EACH RECEPTOR FROM:
● INFINITE LINE SOURCES (WORKSHEET 4, SECTION D1)
● FINITE LINE SOURCES (WORKSHEET 5, SECTION D2)
● AREA SOURCES (WORKSHEET 5, SECTION D3)
COMPUTE TOTAL ONE-HOURLY CO CONCENTRATION
(SECTION E)
DETERMINE TOTAL EIGHT-HOURLY AVERAGE CO CONCENTRATION
(SECTION E)
END

Figure 1. Chapter III procedure for determining ambient CO concentrations in the vicinity of indirect sources.

worst-case eight-hourly CO concentration. Both the one- and eight-hourly procedures consider a variety of degrees of available CO monitoring data, e.g., historical background monitors and local, two-week special monitoring studies.

B. Data Requirements

The data base necessary to formulate inputs to the procedure for determining traffic, emissions, and air quality impacts consists of the following generalized requirements:

- A scaled map of the indirect source, including associated roadways, intersections, parking lots, access roads, internal parking lot traffic links, and so forth.
- Traffic engineering characteristics of each road to be analyzed, i.e., number of lanes, road width, turning channels, type of intersection control, signal timing, percent trucks and buses, and design speed.
- Through and turning traffic volumes for each road.
- Characteristics of the parking area, such as facility emptying time, number of cars per stall, gate capacity, and parking lot capacity.
- The number of trips (as a percentage) attracted to and from the facility through each access gate.
- Background and local air quality measurements.
- Yearly surface and upper-air meteorological data for the area.
- Miscellaneous demographic data, such as metropolitan population, and diurnal roadway traffic patterns.

Specific data required in applying the analysis procedure are listed in Worksheet 1 with guidance given in the instructions following. Appendix A provides methods for generating various traffic parameters for: (1) roadways, (2) airports, (3) shopping centers, (4) sports complexes, (5) municipal parking lots, (6) amusement parks, and (7) recreation areas.

C. Estimation of Emission Rates

Three methods are discussed here for the prediction of the CO emission rates from line and area sources. The first method is used to predict emissions from uninterrupted or freely flowing vehicular traffic (Qf, g m^{-1} s^{-1}). The second method predicts the "excess" emissions (Qe, g m^{-1} s^{-1}), and the distance (Le) over which they occur. Excess emissions are defined as those emissions--attributable to deceleration, idling and acceleration--caused by interruptions to the traffic flow that are in excess of those for average cruise conditions. The third method predicts emissions within an area source (Qa, g m^{-2} s^{-1}) and includes a procedure for predicting emissions caused by queueing when demand exceeds capacity within the area source. All emissions in these Indirect Source Guidelines are based on the updated (December, 1977) Model Emissions Model (Kunselman, 1974). Adjustment factors for a base year of 1977, other calendar years, and a cold-start-hot-start-speed-temperature correction are found using AP-42, Mobile Source Emission Factors (1978). The ratio of light-duty vehicles, light-duty trucks, heavy duty trucks, and motorcycles is variable throughout the guideline (see rationale Step 4, Instructions for Worksheet 1).

The first methodology (Worksheet 2) is used for every line source analyzed (including those within parking areas), while the third methodology (Worksheet 3) is used for every area source analyzed. The second methodology (Worksheet 2) is used to compute excess emissions due to special conditions on a line source (e.g., intersections) or within an area source; these are then considered in addition to the line source analysis for free-flow conditions. In general, the user will determine line source emissions from each direction of traffic flow in a three-step procedure:

1. Compute the line source emission rate that corresponds to a vehicle traveling at an average speed.
2. Compute the excess line source emission rate due to interruptions in traffic flow (due to a signal, stop sign, or other cause) and the distance over which the excess emissions apply.
3. Apply emission adjustment factors (hot-to-cold start ratio, temperature, altitude, calendar year) to obtain the emission rate required for the dispersion analysis.

When an area source is being analyzed, line sources located within the area source are handled in the same manner as any other line source. Worksheet 2 provides a series of sequential steps to compute line source emission rates. Worksheet 3 provides the steps to compute area source emission rates.

It should be noted that the methodologies presented do not, with the exception of the third methodology (parking areas) apply when demand volume exceeds or equals the capacity of a facility. When such a situation is found to exist, a more detailed analysis than can be presented in this guideline format must be undertaken. A methodology which might be used in determining the effects of vehicle traffic when demand exceeds capacity is contained in the National Cooperative Highway Research Program 133, Appendix B (see references).

1. Uninterrupted Flow

This methodology is used to determine the emission rate on roadway sections having uninterrupted or freely flowing traffic, e.g., freeway, expressway, midblock section of a public roadway, or well-defined roadway within a parking lot. Worksheet 2 summarizes all steps in the emissions calculation procedure. The first step in the methodology is to determine the demand volume (V) and free-flow capacity (C) of the road segment. The demand volume is a required input traffic parameter as identified in Worksheet 1; free-flow capacity can be estimated, for example, using Appendix B. The ratio V/C is then determined and used to enter Figures 2-5 to find average cruise speed on each road segment. The figures associated with each road type are as follows:

- Freeways (Figure 2)
- Multilane rural highways (Figure 3)
- Two-lane rural highways (Figure 4)
- Urban arterial streets (Figure 5).

The user enters each figure with a V/C ratio intersected with the appropriate curve for the road type to determine cruise speed. The user should also note that the three curves in Figure 5 are for typical signal progressions, but that the figure does not include curves for every possible arterial speed. To use Figure 5 for other speeds the user should calculate a "cruise" speed by adding the speed limit differential between the subject arterial and the appropriate curve, to the graphically determined speed. (For example, an "uncoordinated arterial" with a speed limit of 40 mph and a V/C ratio of 0.5 requires use of Curve II. The speed limit differential is 40 mph- 25 mph, or 15 mph. This 15 mph differential is now added to the 20 mph cruise speed read from Curve II to give an interpolated average speed of 35 mph). Local estimating techniques may be substituted for the above procedure if available.

Figure 6 illustrates the variation of the emissions (Ef, g veh^{-1} m^{-1}) for a single vehicle as a function of vehicle speed. This value (i.e., Ef) multiplied by the vehicle flow rate (veh hr^{-1}) determines the free flow emission rate (Qf):

$$Qf = \frac{Ef\ .\ V}{3600}\ .(gm\ m^{-1}\ sec^{-1}) \qquad (1)$$

The emission rate calculated in Eq. (1) is a reference value appropriate to vehicle emission rates for: (1) a given reference year (1977) and (2) specified ambient characteristics (e.g., temperature). Thus, the actual emission factors vary depending on vehicle type, calendar year, catalyst or non-catalyst, altitude, State (California or elsewhere), ambient temperature, the percentage of cold-starting vehicles, the percentage of hot starting vehicles, and vehicle speed. A total correction factor (C_T) for these variables is discussed in the Worksheet 2 instructions and may be calculated using Tables 1, 2, and 3. These tables are set up for several combinations of the above variables, but cannot be all inclusive. Interpolation and extrapolation may be used to expand table corrections or the Mobile Source Emission Factors (1978) document may be consulted.

The actual emission rate (Qf') is then determined as the product of Qf (Eq. [1]) and the total emission correction factor:

$$Qf' = (Qf)(C_T) \qquad (2)$$

Emission rates must be calculated for each roadway analyzed. Freeways and expressways require no further emission calculations. Signalized and signed intersection approaches (interrupted flow), however, require the additional computation of "excess" emissions due to the acceleration, deceleration and idling of vehicles that stop at the intersection.

2. Interrupted Flow

a. Scope

The determination of the emission rate for situations of interrupted traffic flow (i.e., at toll booths, and signed and signalized intersections) involves the determination of so-called "excess" emissions. These are vehicular emissions that are generated over a finite segment of roadway as a result of idling, acceleration, and deceleration; they represent the excess emissions in that region beyond that which an equal number of freely flowing vehicles would emit at cruise. The determination of excess emissions requires the application of a nine-part procedure:

- Specification of site specific traffic and engineering parameters.
- Computation of derived intersection parameters (i.e., green-phase capacity and service volume).
- Determination of the proportion of stopping vehicles.
- Determination of the number of vehicles subject to queueing delay.
- Determination of maximum queue length.
- Computation of excess vehicle running time.

- Computation of effective distance for excess emissions.
- Computation of excess emissions.
- Application of a total emission correction factor.

The step-by-step computational sequence is given in Worksheet 2 with the following subsections discussing steps in the procedure.

b. Vehicle Queueing at Signalized Intersections

The parameters of one-hour demand volume, approach width, percentage left- and right-turning vehicles, percentages of trucks and buses, and metropolitan area size are defined inputs of Worksheet 1. These parameters are used to determine capacity service volume per hour of green (Cs) and capacity (C) using Worksheet B (Appendix B). Alternate methods, such as the Highway Capacity Manual (1965) or the Leisch nomographs (1967), may also be used to determine capacity. Worksheet 2 permits the assignment of left turn green-phase as well as through traffic green-phase to each intersection approach. Therefore, turning channels can be handled uniquely from through and right-turn lanes. When two signal phases control one approach, the green-phase capacity must be determined for left-turn as well as through signal phases (see example 2, Chapter V).

The proportion (P) of vehicles that stop for a signal is determined using an equation developed by Webster (1958):

$$P = \frac{1-G/Cy}{1-V/Cs} \qquad (3)$$

where Cy is the length of the signal cycle (Worksheet B), G is the effective green-phase time (Worksheet B), V is the 1-hour traffic demand for an approach, and Cs is the capacity service volume for each approach per 1-hour of green time. Note that if P is greater than 1.0, these guidelines are not appropriate for determining traffic flow (and hence air quality) at this intersection. The user is referred to the National Cooperative Highway Research Program 133 (1972) for possible alternative methods of predicting delay at an over-capacity intersection.

The number (N) of vehicles subject to queueing delay per cycle is:

$$N = \frac{P\ V\ Cy}{3600} + \frac{V}{C-V}\ , \tag{4}$$

where $\frac{P\ V\ Cy}{3600}$ is the number that must stop on any cycle

$\frac{V}{C-V}$ is the "residual" number that remains in the queue at the end of the green phase.

$C = Cs\ G/Cy$, and is the actual capacity per hour.

The maximum length of the queue (Lq, m) is:

$$Lq = 8N/M$$

where 8 is the distance (meters) occupied by each queued vehicle and M is the number of lanes in the approach. Then the average excess running time, Rq (also called queued delay) for all vehicles traveling through the intersection is:

$$Rq = 0.5\ P(Cy-G) + \frac{3600\ V}{C(C-V)}\ . \tag{5}$$

Equation (5) is adapted from concepts developed by Newell (1965) concerning vehicles' delay at approaches to signalized intersections. If more than one phase controls an approach, the volumes handled by each phase are proportioned to the total volume to weight each phase's running time accordingly. (See instruction 9, Worksheet 2).

c. Vehicle Queueing at Signed Intersections and Toll Booths

The methodology used here is different from that used at signalized intersections in that all vehicles must stop and wait to be served by the intersection or toll booth. The average number of vehicles waiting to leave the intersection is computed based on classical queueing theory by using the following equation:

$$N = \frac{V}{C-V} \qquad (6)$$

The length of the queue in meters is:

$$Lq = \frac{8N}{M} \qquad (7)$$

The average* excess running time for vehicles at a nonsignalized intersection is the queue length (expressed here in number of vehicles) multiplied by the average time lapse between successive vehicles entering the intersection (which can be expressed by $\frac{1}{C}$. For a nonsignalized intersection, Rq, is determined as follows:

$$Rq = \frac{3600\ N}{C}\ . \qquad (8)$$

d. Excess Emission Rate

The computation of excess emissions is the same for either signalized or nonsignalized intersections. The variable elements required for computation are: queued delay (Rq), proportion of stopped vehicles (P), the length of the queue (Lq), and the initial and target speeds (S) of vehicles entering and leaving the queue, respectively. Excess emissions are calculated for two modes of vehicle operation: (1) acceleration-deceleration and (2) idling.

The length of the roadway (Lad) required for acceleration or deceleration of a single vehicle is given by:

---

*Average refers to the hourly mean for all vehicles using the intersection.

$$\mathrm{Lad} = \frac{1}{2}\frac{S^2}{A'} = \frac{1}{2}\frac{S^2}{2.5}\,\frac{1609}{3600} = 0.0894S^2 \text{ (m)} \qquad (9)$$

where $A' = 2.5$ is the assumed acceleration and deceleration rate (mph s^{-1})

1609 is a conversion factor from miles to meters

3600 is a conversion factor from hours to seconds

S is the speed (mph) from which deceleration began or to which the vehicle accelerates.

The total excess emissions are assumed to be uniform over a specified length of roadway, Le. Usually, idling emissions are the largest contribution to the total excess emissions, however, if the queue is short, then accelerating and decelerating vehicles are the largest contribution to the total excess emissions. Accordingly, the effective excess emissions length (Le) is taken to be the larger of two values: Lq, the length of the vehicle queue; or Lad = 40, the distance required to decelerate from and accelerate to a cruise speed of 15 mph.* Therefore:

$$\mathrm{Le} = \mathrm{Max}\,(\mathrm{Lq},\ 40)\ . \qquad (10)$$

Excess emission values for the accelerating and decelerating modes are determined using Figure 7. Figure 7 is derived from the 1977 updated modal analysis previously described by Kunselman et al. (1974). In the figure, the initial and target (i.e., steady) speed emissions have been subtracted to provide only the excess emissions (Ea for acceleration, and Ed for deceleration--g veh-m^{-1}). The excess emission rate (Qad, for acceleration/deceleration) per stopping/starting vehicle is then determined as:

$$\mathrm{Qad} = (\mathrm{Ea} + \mathrm{Ed})\ P\ V/3600 \ (\mathrm{gm\ m^{-1}\ sec^{-1}}) \qquad (11)$$

---

*If the initial and target speeds or acceleration/deceleration rates differ significantly from the assumed values of 15 mph and 2.5 mph s^{-1}, respectively, then a corresponding new minimum Le should be substituted (Eq. [9]) for the 40-m value used here.

The method of computing idling emissions (Patterson and Meyer 1975) is based on the estimated excess running time (Rq) for intersections. But Rq includes both vehicle idling time as well as "lost" time due to acceleration and deceleration. The excess emission rate for idling vehicles is determined as the product of: (1) the average idling time, (2) the average idling emission rate per vehicle, and (3) the vehicle flow rate, divided by (4) the length of the queue:

$$Qs = \frac{0.42\ (Rq - 0.2S)(V/3600)}{Lq}\ (\text{gm sec}^{-1}\ \text{m}^{-1}) \qquad (12)$$

In Eq. (12), the idling time is computed as the difference between the average delay (Rq) and the acceleration/deceleration portion of delay (assumed equal to 0.2S). The CO idle emission rate is 0.42 g $\text{veh}^{-1}\ \text{s}^{-1}$ for a 1977 emissions scenario at idle speed, 100% light duty vehicles, 0% cold starts, 0% hot starts, 75°F, non-California and low altitude.

The weighted average of the excess emission rate (Qe) over the roadway length, Le, is then determined as follows:

$$Qe = \frac{Qs\ Lq + Qad\ Lad}{Le}\ . \qquad (13)$$

A total correction factor C_T may then be applied to the weighted average excess emission rate (Qe) to determine the actual excess emission rate (Qe') for the effective roadway length (Le):

$$Qe' = Qe\ C_T \qquad (14)$$

The Qe' and Le values are then used as inputs to the dispersion methodology in determining ambient CO concentrations at specified receptor locations (Subsection III-D).

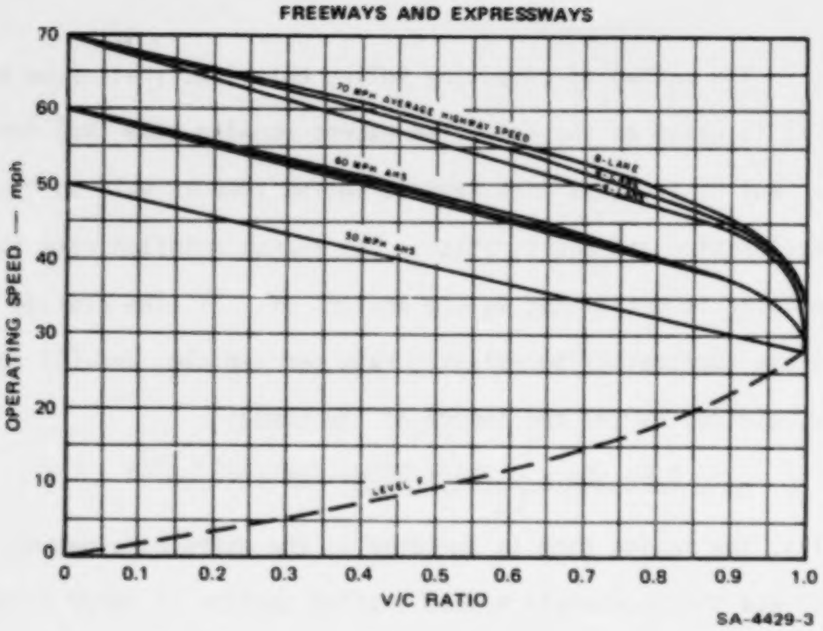

FIGURE 2 RELATIONSHIPS BETWEEN V/C RATIO AND OPERATING SPEED, IN ONE DIRECTION OF TRAVEL, ON FREEWAYS AND EXPRESSWAYS, UNDER UNINTERRUPTED FLOW CONDITIONS

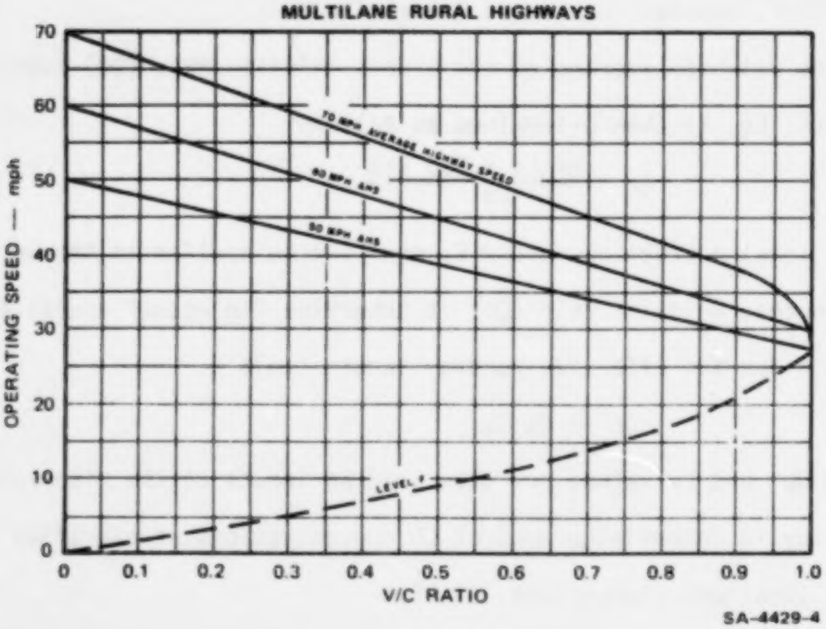

FIGURE 3 RELATIONSHIPS BETWEEN V/C RATIO AND OPERATING SPEED, IN ONE DIRECTION OF TRAVEL, ON MULTILANE RURAL HIGHWAYS, UNDER UNINTERRUPTED FLOW CONDITIONS

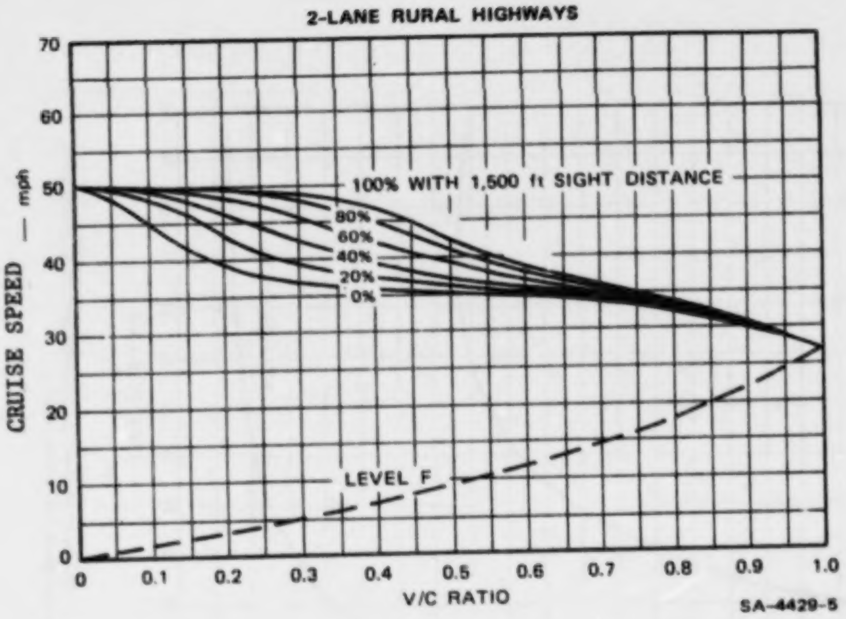

FIGURE 4 RELATIONSHIPS BETWEEN V/C RATIO AND OPERATING SPEED, OVERALL FOR BOTH DIRECTIONS OF TRAVEL, ON TWO-LANE RURAL HIGHWAYS WITH AVERAGE HIGHWAY SPEED OF 50 MPH, UNDER UNINTERRUPTED FLOW CONDITIONS

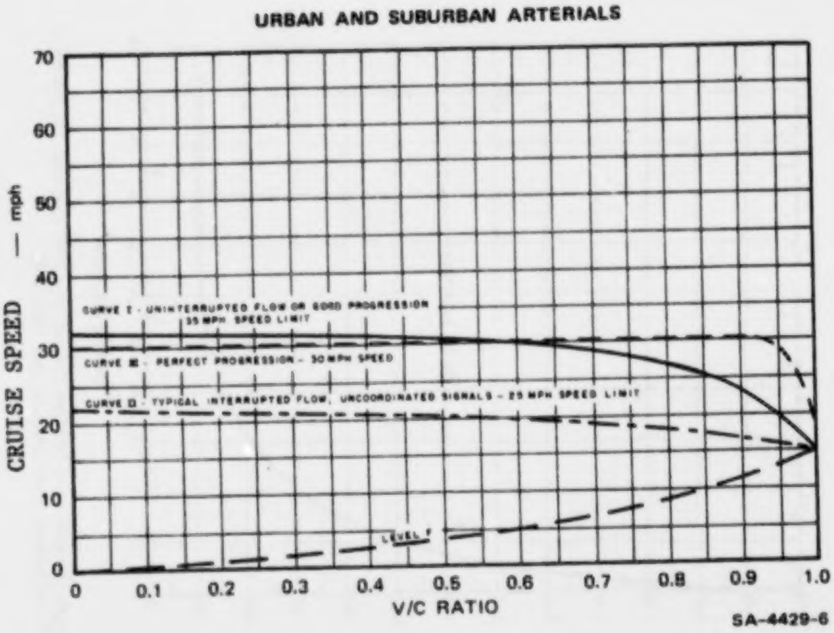

FIGURE 5 TYPICAL RELATIONSHIPS BETWEEN V/C RATIO AND AVERAGE OVERALL TRAVEL SPEED, IN ONE DIRECTION OF TRAVEL, ON URBAN AND SUBURBAN ARTERIAL STREETS

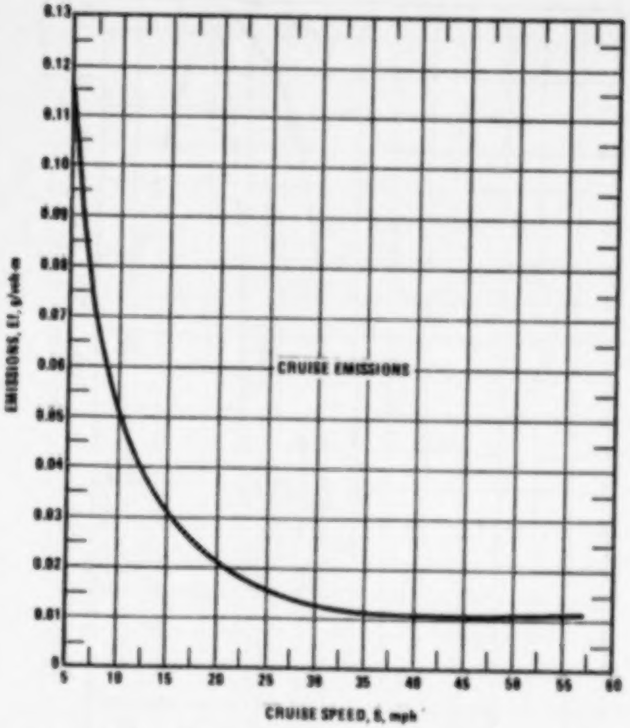

Figure 6. Emissions as a function of speed for a 1977 base year, 0% cold starts, 75°F, low-altitude, non-California, and 100% LDV.

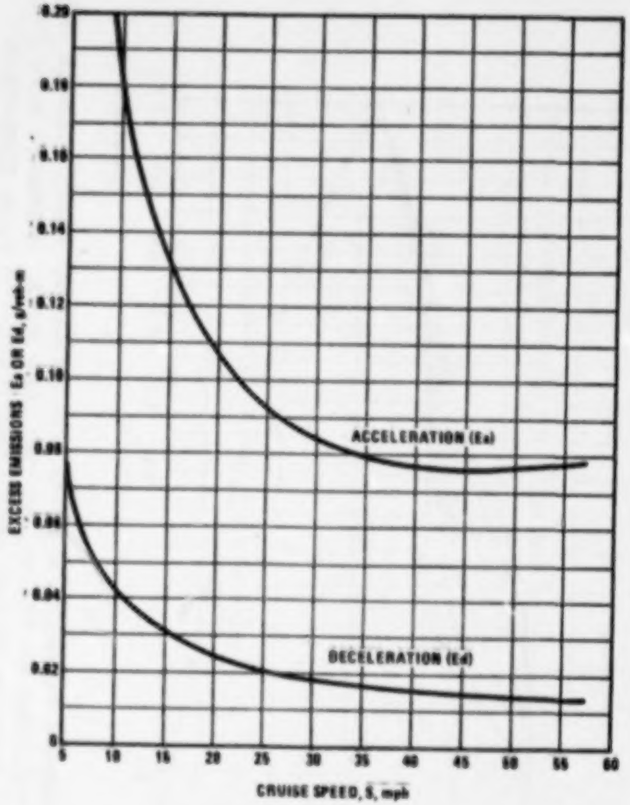

Figure 7. Excess emissions due to acceleration to or deceleration from cruise speed (1977 base year).

EMISSION CORRECTION FACTORS FOR REGION: LOW ALTITUDE

			YEAR:	1978	1978	1978	1978	1980	1980	1980	1980	1982	1982	1982	1982
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M**	H	C	T												
LDV	20	10	20	1.22	1.30	1.37	1.42	1.00	1.11	1.17	1.22	.79	.83	.89	.93
	20	10	40	1.12	1.18	1.23	1.26	.91	.92	1.04	1.08	.67	.75	.79	.82
	20	35	20	1.94	2.17	2.39	2.59	1.49	1.80	2.09	2.25	1.25	1.43	1.59	1.72
	20	35	40	1.57	1.74	1.90	2.03	1.31	1.47	1.63	1.75	1.00	1.13	1.25	1.35
	20	60	20	2.65	3.03	3.42	3.74	2.28	2.66	3.00	3.29	1.75	2.03	2.29	2.51
	20	60	40	2.03	2.30	2.57	2.81	1.72	1.98	2.23	2.43	1.33	1.52	1.71	1.87
LDT	20	10	20	1.58	1.59	1.67	1.75	1.43	1.50	1.59	1.67	1.25	1.42	1.57	1.60
	20	10	40	1.47	1.46	1.53	1.59	1.32	1.37	1.44	1.51	1.15	1.28	1.36	1.42
	20	35	20	2.40	2.54	2.82	3.05	2.19	2.46	2.73	2.95	1.97	2.38	2.65	2.87
	20	35	40	2.01	2.11	2.31	2.48	1.83	2.00	2.20	2.37	1.61	1.90	2.10	2.26
	20	60	20	3.21	3.53	3.92	4.35	2.95	3.42	3.86	4.23	2.69	3.35	3.79	4.15
	20	60	40	2.55	2.76	3.09	3.38	2.33	2.63	2.96	3.23	2.08	2.51	2.83	3.10
MC	20	10	20	.47	.81	.92	1.02	.38	.67	.72	.77	.30	.54	.57	.60
	20	10	40	.42	.73	.83	.92	.34	.59	.64	.68	.27	.48	.50	.52
	20	35	20	.78	1.36	1.55	1.71	.63	1.17	1.31	1.43	.51	.98	1.08	1.17
	20	35	40	.62	1.08	1.23	1.35	.50	.91	1.01	1.10	.40	.75	.82	.89
	20	60	20	1.09	1.90	2.17	2.40	.89	1.49	1.80	2.00	.72	1.41	1.60	1.79
	20	60	40	.81	1.42	1.67	1.79	.66	1.23	1.39	1.52	.53	1.07	1.15	1.26

			YEAR:	1978	1978	1978	1978	1980	1980	1980	1980	1982	1982	1982	1982
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M	H	C	T												
LDV	20	10	60	1.05	1.10	1.14	1.16	.85	.92	.96	.99	.63	.70	.74	.76
	20	10	80	1.01	1.05	1.08	1.10	.81	.88	.91	.93	.60	.67	.70	.72
	20	35	60	1.34	1.46	1.59	1.69	1.11	1.24	1.35	1.44	.85	.95	1.05	1.12
	20	35	80	1.19	1.28	1.38	1.46	.98	1.09	1.18	1.25	.75	.84	.92	.98
	20	60	60	1.63	1.83	2.04	2.21	1.37	1.56	1.75	1.90	1.07	1.21	1.36	1.48
	20	60	80	1.37	1.52	1.68	1.82	1.15	1.30	1.45	1.57	.90	1.02	1.14	1.24
LDT	20	10	60	1.40	1.37	1.43	1.48	1.25	1.28	1.34	1.40	1.08	1.19	1.26	1.31
	20	10	80	1.34	1.31	1.36	1.40	1.21	1.23	1.28	1.33	1.04	1.14	1.20	1.24
	20	35	60	1.75	1.80	1.94	2.10	1.58	1.70	1.86	1.99	1.39	1.60	1.75	1.88
	20	35	80	1.56	1.59	1.72	1.83	1.42	1.50	1.63	1.74	1.24	1.41	1.54	1.64
	20	60	60	2.10	2.24	2.50	2.72	1.91	2.12	2.37	2.58	1.69	2.00	2.24	2.44
	20	60	80	1.77	1.87	2.08	2.26	1.62	1.78	1.98	2.15	1.44	1.68	1.87	2.03
MC	20	10	60	.39	.67	.77	.84	.31	.54	.58	.61	.24	.43	.44	.46
	20	10	80	.37	.63	.72	.79	.29	.50	.53	.56	.23	.40	.40	.41
	20	35	60	.50	.87	.99	1.10	.40	.72	.79	.86	.31	.58	.63	.68
	20	35	80	.42	.73	.83	.91	.33	.58	.63	.68	.25	.46	.50	.52
	20	60	60	.61	1.07	1.22	1.35	.49	.90	1.01	1.10	.39	.74	.82	.90
	20	60	80	.47	.82	.93	1.03	.36	.66	.74	.80	.28	.51	.59	.63
HDG				1.72	5.80	5.51	6.36	1.73	5.54	5.62	6.56	1.73	5.23	5.77	6.91
HDD				.03	.62	.60	.62	.03	.60	.57	.57	.03	.60	.55	.53

**LDV-light duty vehicles, LDT-light duty trucks, MC-motorcycles, HDG-heavy duty gas trucks, HDD-heavy duty diesel.

Table 1a. Total emission correction factor (C_T) for low altitude by calendar year, vehicle speed (mph), vehicle type (M), % hot starts (H), % cold starts (C), and temperature (T-°F).

EMISSION CORRECTION FACTORS FOR REGION: LOW ALTITUDE

			YEAR:	1985	1985	1985	1985	1987	1987	1987	1987	1990	1990	1990	1990
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M**	H	C	T												
LDV	20	10	20	.47	.52	.57	.60	.38	.40	.44	.46	.31	.30	.33	.35
	20	10	40	.43	.47	.51	.54	.35	.37	.40	.42	.29	.28	.31	.32
	20	35	20	.85	.92	1.04	1.13	.71	.77	.81	.88	.61	.57	.64	.69
	20	35	40	.69	.75	.84	.90	.59	.60	.67	.73	.52	.49	.55	.60
	20	60	20	1.22	1.33	1.50	1.65	1.03	1.04	1.19	1.30	.90	.83	.94	1.04
	20	60	40	.96	1.03	1.16	1.27	.84	.84	.95	1.04	.75	.70	.79	.87
LDT	20	10	20	.97	1.19	1.27	1.34	.76	.99	1.07	1.13	.52	.76	.83	.87
	20	10	40	.88	1.06	1.14	1.19	.70	.89	.96	1.00	.47	.68	.74	.77
	20	35	20	1.55	2.00	2.23	2.42	1.22	1.68	1.88	2.03	.83	1.30	1.46	1.58
	20	35	40	1.26	1.58	1.75	1.89	.99	1.33	1.47	1.59	.67	1.02	1.14	1.23
	20	60	20	2.13	2.81	3.18	3.49	1.68	2.37	2.68	2.94	1.15	1.84	2.09	2.29
	20	60	40	1.63	2.10	2.36	2.58	1.29	1.76	1.94	2.17	.88	1.36	1.54	1.68
MC	20	10	20	.15	.27	.28	.29	.09	.18	.18	.19	.07	.12	.13	.13
	20	10	40	.13	.24	.25	.25	.08	.15	.16	.16	.06	.11	.11	.11
	20	35	20	.25	.49	.54	.58	.16	.32	.35	.38	.11	.23	.25	.27
	20	35	40	.20	.38	.41	.44	.13	.24	.26	.28	.09	.17	.19	.20
	20	60	20	.36	.71	.80	.87	.23	.46	.52	.57	.16	.33	.37	.41
	20	60	40	.26	.51	.57	.63	.17	.33	.37	.41	.12	.24	.27	.29

			YEAR:	1985	1985	1985	1985	1987	1987	1987	1987	1990	1990	1990	1990
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M	H	C	T												
LDV	20	10	60	.40	.44	.48	.50	.33	.34	.37	.39	.27	.26	.29	.31
	20	10	80	.39	.42	.46	.48	.31	.33	.36	.37	.26	.25	.28	.29
	20	35	60	.60	.64	.72	.77	.52	.53	.59	.63	.47	.44	.49	.53
	20	35	80	.54	.58	.64	.69	.47	.48	.53	.57	.43	.40	.45	.48
	20	60	60	.79	.84	.95	1.04	.71	.71	.80	.88	.66	.61	.69	.75
	20	60	80	.69	.73	.82	.90	.63	.63	.71	.77	.59	.55	.62	.68
LDT	20	10	60	.83	.97	1.05	1.10	.66	.83	.89	.93	.44	.64	.68	.71
	20	10	80	.80	.95	1.00	1.04	.63	.80	.95	.88	.41	.61	.65	.69
	20	35	60	1.08	1.33	1.46	1.57	.86	1.12	1.23	1.37	.58	.86	.96	1.02
	20	35	80	.97	1.18	1.27	1.39	.77	.99	1.09	1.16	.52	.76	.84	.90
	20	60	60	1.32	1.67	1.87	2.04	1.05	1.40	1.58	1.72	.72	1.08	1.21	1.32
	20	60	80	1.13	1.41	1.57	1.71	.90	1.19	1.33	1.45	.62	.91	1.02	1.11
MC	20	10	60	.12	.22	.22	.22	.08	.14	.14	.14	.05	.10	.10	.10
	20	10	80	.11	.20	.20	.20	.07	.13	.13	.13	.05	.09	.09	.09
	20	35	60	.16	.29	.32	.34	.10	.19	.20	.21	.07	.13	.14	.15
	20	35	80	.13	.23	.25	.26	.08	.15	.16	.16	.05	.11	.11	.11
	20	60	60	.19	.37	.41	.45	.12	.24	.27	.29	.08	.17	.19	.20
	20	60	80	.14	.27	.30	.32	.09	.17	.19	.20	.06	.12	.13	.14
HDG				1.57	3.97	4.67	5.69	1.52	3.21	3.91	4.78	1.55	2.52	3.20	3.95
HDD				.03	.59	.53	.51	.03	.59	.52	.51	.03	.59	.52	.50

**LDV-light duty vehicles, LDT-light duty trucks, MC-motorcycles, HDG-heavy duty gas trucks, HDD-heavy-duty diesel.

Table 1b. Total emission correction factor (C_T) for low altitude by calendar year, vehicle speed (mph), vehicle type (M), % hot starts (H), % cold starts (C), and temperature (T-°F).

EMISSION CORRECTION FACTORS FOR REGION: CALIFORNIA

			YEAR:	1985	1985	1985	1985	1987	1987	1987	1987	1990	1990	1990	1990
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M**	H	C	T												
LDV	20	10	20	.18	.40	.44	.46	.11	.35	.38	.40	.06	.31	.34	.36
	20	10	40	.17	.38	.41	.43	.10	.32	.35	.37	.06	.29	.32	.33
	20	35	20	.28	.73	.82	.89	.17	.65	.73	.79	.11	.59	.67	.73
	20	35	40	.24	.64	.72	.78	.15	.57	.64	.69	.10	.52	.58	.63
	20	60	20	.38	1.06	1.21	1.32	.24	.95	1.08	1.19	.16	.87	.99	1.09
	20	60	40	.32	.91	1.03	1.12	.20	.81	.92	1.01	.13	.75	.85	.93
LDT	20	10	20	.81	.85	.88	.90	.64	.70	.74	.75	.45	.55	.59	.61
	20	10	40	.77	.80	.83	.84	.60	.66	.69	.70	.43	.52	.55	.57
	20	35	20	1.32	1.42	1.57	1.68	1.09	1.20	1.32	1.42	.77	.97	1.08	1.17
	20	35	40	1.16	1.25	1.37	1.46	.93	1.06	1.16	1.24	.69	.86	.98	1.07
	20	60	20	1.82	2.00	2.24	2.45	1.45	1.70	1.91	2.08	1.09	1.39	1.58	1.72
	20	60	40	1.56	1.71	1.91	2.08	1.25	1.46	1.63	1.78	.94	1.20	1.36	1.48
MC	20	10	20	.26	.29	.31	.32	.17	.18	.20	.20	.12	.13	.13	.14
	20	10	40	.23	.25	.27	.28	.15	.16	.17	.17	.11	.11	.12	.12
	20	35	20	.47	.52	.58	.63	.32	.34	.37	.40	.23	.24	.26	.28
	20	35	40	.36	.40	.44	.48	.24	.26	.28	.30	.17	.18	.20	.21
	20	60	20	.69	.75	.85	.94	.46	.49	.55	.60	.33	.34	.39	.43
	20	60	40	.50	.55	.62	.67	.33	.35	.40	.43	.24	.25	.28	.30

			YEAR:	1985	1985	1985	1985	1987	1987	1987	1987	1990	1990	1990	1990
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M	H	C	T												
LDV	20	10	60	.16	.36	.39	.40	.10	.31	.33	.35	.06	.27	.30	.31
	20	10	80	.15	.35	.37	.39	.09	.30	.32	.34	.05	.26	.29	.30
	20	35	60	.22	.58	.64	.69	.14	.51	.57	.62	.09	.47	.52	.57
	20	35	80	.20	.53	.59	.63	.13	.47	.53	.57	.08	.43	.48	.52
	20	60	60	.28	.79	.90	.98	.18	.72	.81	.89	.12	.66	.75	.82
	20	60	80	.25	.72	.81	.88	.16	.65	.73	.80	.11	.60	.68	.74
LDT	20	10	60	.74	.76	.79	.79	.58	.63	.66	.67	.41	.50	.53	.54
	20	10	80	.72	.74	.76	.76	.56	.61	.63	.64	.40	.48	.51	.52
	20	35	60	1.05	1.13	1.23	1.30	.84	.96	1.05	1.12	.63	.78	.86	.93
	20	35	80	.98	1.04	1.13	1.19	.78	.89	.97	1.02	.58	.73	.80	.85
	20	60	60	1.37	1.50	1.67	1.81	1.11	1.28	1.44	1.56	.84	1.07	1.20	1.31
	20	60	80	1.24	1.35	1.50	1.62	1.00	1.16	1.30	1.41	.77	.97	1.09	1.19
MC	20	10	60	.21	.23	.24	.25	.14	.15	.15	.15	.10	.10	.10	.10
	20	10	80	.19	.21	.22	.22	.13	.13	.14	.14	.09	.09	.09	.09
	20	35	60	.28	.31	.34	.36	.19	.20	.22	.23	.13	.14	.15	.16
	20	35	80	.23	.25	.27	.28	.15	.16	.17	.18	.10	.11	.12	.12
	20	60	60	.36	.40	.44	.48	.24	.25	.29	.31	.17	.18	.20	.21
	20	60	80	.26	.29	.32	.34	.17	.18	.20	.22	.12	.13	.14	.15
HDG				1.19	3.77	4.81	6.04	1.25	3.09	4.00	5.01	1.38	2.45	3.27	4.11
HDD				.03	.60	.52	.51	.03	.59	.52	.50	.03	.59	.52	.50

**LDV-light duty vehicles, LDT-light duty trucks, MC-motorcycles, HDG-heavy duty gas trucks, HDD-heavy duty diesel.

Table 2a. Total emission correction factor (C_T) for California by calendar year, vehicle speed (mph), vehicle type (M), % hot starts (H), % cold starts (C), and temperature (T-°F).

EMISSION CORRECTION FACTORS FOR REGION: CALIFORNIA

			YEAR:	1978	1978	1978	1978	1980	1980	1980	1980	1982	1982	1982	1982
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M**	H	C	T												
LDV	20	10	20	1.09	1.07	1.11	1.14	.79	.91	.85	.87	.44	.60	.63	.65
	20	10	40	1.02	1.00	1.02	1.04	.69	.76	.79	.80	.41	.56	.59	.61
	20	35	20	1.69	1.71	1.87	2.01	1.10	1.33	1.46	1.57	.66	1.01	1.12	1.21
	20	35	40	1.41	1.45	1.58	1.68	.96	1.19	1.25	1.33	.58	.88	.97	1.09
	20	60	20	2.19	2.35	2.69	2.97	1.47	1.89	2.07	2.26	.88	1.43	1.61	1.76
	20	60	40	1.79	1.91	2.14	2.32	1.22	1.52	1.71	1.86	.79	1.20	1.35	1.47
LDT	20	10	20	1.62	1.53	1.55	1.56	1.41	1.35	1.38	1.39	1.16	1.13	1.17	1.19
	20	10	40	1.51	1.42	1.43	1.42	1.32	1.26	1.28	1.29	1.09	1.06	1.09	1.10
	20	35	20	2.53	2.46	2.67	2.89	2.23	2.29	2.40	2.56	1.85	1.87	2.05	2.19
	20	35	40	2.15	2.08	2.24	2.36	4.92	1.89	2.04	2.16	1.62	1.63	1.77	1.88
	20	60	20	3.43	3.39	3.79	4.12	3.05	3.05	3.42	3.72	2.55	2.61	2.93	3.19
	20	60	40	2.79	2.75	3.05	3.31	2.52	2.51	2.80	3.04	2.14	2.19	2.45	2.66
MC	20	10	20	.66	.83	.95	1.04	.56	.69	.76	.81	.48	.56	.61	.69
	20	10	40	.60	.75	.85	.94	.50	.61	.67	.71	.42	.49	.53	.55
	20	35	20	1.10	1.39	1.58	1.75	.99	1.21	1.36	1.49	.87	1.02	1.19	1.29
	20	35	40	.87	1.10	1.26	1.39	.77	.94	1.05	1.14	.67	.78	.87	.94
	20	60	20	1.54	1.99	2.27	2.46	1.42	1.73	1.97	2.17	1.26	1.48	1.67	1.89
	20	60	40	1.15	1.45	1.66	1.83	1.09	1.26	1.43	1.57	.91	1.07	1.21	1.32

			YEAR:	1978	1978	1978	1978	1980	1980	1980	1980	1982	1982	1982	1982
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M	H	C	T												
LDV	20	10	60	.98	.99	.96	.98	.66	.72	.74	.75	.40	.53	.56	.57
	20	10	80	.94	.91	.92	.93	.64	.69	.71	.72	.38	.51	.53	.55
	20	35	60	1.24	1.27	1.37	1.45	.85	1.01	1.09	1.16	.51	.78	.86	.92
	20	35	80	1.12	1.19	1.22	1.28	.77	.91	.98	1.04	.47	.72	.78	.83
	20	60	60	1.50	1.60	1.78	1.92	1.03	1.30	1.45	1.57	.63	1.04	1.16	1.27
	20	60	80	1.29	1.37	1.52	1.64	.90	1.13	1.24	1.36	.56	.92	1.03	1.12
LDT	20	10	60	1.43	1.39	1.34	1.32	1.25	1.19	1.20	1.20	1.09	1.01	1.03	1.03
	20	10	80	1.37	1.28	1.27	1.25	1.21	1.15	1.15	1.14	1.01	.98	.99	.99
	20	35	60	1.87	1.81	1.93	2.02	1.69	1.66	1.78	1.87	1.49	1.45	1.57	1.65
	20	35	80	1.67	1.61	1.70	1.77	1.53	1.50	1.59	1.67	1.32	1.32	1.42	1.49
	20	60	60	2.32	2.28	2.57	2.72	2.13	2.13	2.36	2.55	1.85	1.89	2.10	2.27
	20	60	80	1.98	1.94	2.14	2.29	1.85	1.85	2.09	2.19	1.69	1.67	1.85	2.00
MC	20	10	60	.55	.69	.79	.86	.45	.55	.60	.64	.38	.45	.47	.49
	20	10	80	.51	.65	.74	.81	.42	.51	.55	.59	.35	.41	.43	.45
	20	35	60	.71	.89	1.07	1.12	.61	.74	.82	.89	.52	.61	.67	.72
	20	35	80	.59	.74	.85	.93	.49	.60	.66	.71	.41	.48	.53	.56
	20	60	60	.87	1.10	1.25	1.38	.76	.93	1.05	1.15	.66	.77	.87	.94
	20	60	80	.67	.89	.96	1.06	.54	.68	.76	.83	.47	.55	.62	.67
HDG				1.44	5.41	5.77	7.04	1.33	5.12	5.90	7.29	1.25	4.89	6.01	7.52
HDD				.03	.63	.57	.58	.03	.61	.54	.54	.03	.60	.53	.52

**LDV-light duty vehicles, LDT-light duty trucks, MC-motorcycles, HDG-heavy duty gas trucks, HDD-heavy duty diesel.

Table 2b. Total emission correction factor (C_T) for California by calendar year, vehicle speed (mph), vehicle type (M), % hot starts (H), % cold starts (C), and temperature (T-°F).

EMISSION CORRECTION FACTORS FOR REGION: HIGH ALTITUDE

M**	H	C	T	YEAR: 1978 SPEED: 0	1978 15	1978 30	1978 45	1980 0	1980 15	1980 30	1980 45	1982 0	1982 15	1982 30	1982 45
LDV	20	10	20	1.04	1.73	2.13	2.44	.85	1.46	1.74	1.98	.65	1.07	1.26	1.40
	20	10	40	.93	1.56	1.93	2.22	.76	1.30	1.58	1.78	.58	.96	1.13	1.26
	20	35	20	1.84	2.94	3.48	3.91	1.53	2.56	2.99	3.33	1.19	1.87	2.17	2.40
	20	35	40	1.48	2.34	2.79	3.14	1.21	2.00	2.35	2.62	.94	1.47	1.71	1.90
	20	60	20	2.65	4.16	4.93	5.38	2.22	3.65	4.22	4.68	1.73	2.67	3.07	3.40
	20	60	40	2.02	3.13	3.65	4.07	1.66	2.69	3.11	3.46	1.30	1.94	2.24	2.54
LDT	20	10	20	1.97	2.20	2.77	3.21	1.71	2.02	2.50	2.87	1.44	1.87	2.26	2.56
	20	10	40	1.79	2.01	2.55	2.97	1.56	1.83	2.29	2.63	1.30	1.68	2.04	2.31
	20	35	20	3.39	3.64	4.35	4.91	2.93	3.39	4.02	4.52	2.98	3.21	3.76	4.20
	20	35	40	2.76	2.98	3.59	4.07	2.39	2.73	3.26	3.68	2.00	2.53	2.99	3.39
	20	60	20	4.81	5.08	5.92	6.61	4.15	4.76	5.53	6.17	3.51	4.55	5.27	5.88
	20	60	40	3.73	3.99	4.62	5.17	3.21	3.43	4.24	4.73	2.69	3.19	3.93	4.37
MC	20	10	20	1.45	1.09	1.39	1.63	.88	.88	1.12	1.32	.58	.70	.90	1.05
	20	10	40	1.28	.98	1.26	1.50	.77	.78	1.01	1.19	.50	.62	.80	.98
	20	35	20	2.67	1.89	2.71	2.52	1.66	1.58	1.89	2.19	1.12	1.30	1.55	1.76
	20	35	40	2.08	1.4[illegible]	1.77	2.03	1.28	1.21	1.97	1.69	.85	.99	1.20	1.37
	20	60	20	3.40	2.59	3.03	3.40	2.44	2.27	2.65	2.96	1.66	1.89	2.21	2.47
	20	60	40	2.88	1.93	2.27	2.56	1.79	1.65	1.99	2.18	1.20	1.36	1.60	1.79

M	H	C	T	YEAR: 1978 SPEED: 0	1978 15	1978 30	1978 45	1980 0	1980 15	1980 30	1980 45	1982 0	1982 15	1982 30	1982 45
LDV	20	10	60	.87	1.45	1.81	2.08	.70	1.21	1.46	1.65	.54	.89	1.05	1.18
	20	10	80	.82	1.37	1.73	1.99	.66	1.14	1.39	1.58	.51	.85	1.00	1.18
	20	35	60	1.29	1.96	2.35	2.66	1.01	1.65	1.95	2.18	.79	1.23	1.44	1.69
	20	35	80	1.08	1.71	2.06	2.34	.88	1.44	1.71	1.91	.67	1.08	1.27	1.40
	20	60	60	1.61	2.47	2.89	3.23	1.31	2.10	2.48	2.71	1.09	1.57	1.82	2.01
	20	60	80	1.33	2.04	2.40	2.69	1.09	1.73	2.02	2.25	.88	1.32	1.53	1.69
LDT	20	10	60	1.66	1.89	2.41	2.81	1.45	1.71	2.15	2.48	1.21	1.56	1.90	2.16
	20	10	80	1.57	1.79	2.30	2.69	1.38	1.63	2.06	2.37	1.16	1.48	1.82	2.07
	20	35	60	2.31	2.57	3.07	3.50	2.02	2.30	2.77	3.14	1.69	2.11	2.51	2.81
	20	35	80	2.00	2.21	2.71	3.10	1.76	2.02	2.45	2.78	1.48	1.85	2.21	2.48
	20	60	60	2.97	3.16	3.73	4.19	2.58	2.90	3.40	3.80	2.16	2.67	3.11	3.46
	20	60	80	2.93	2.63	3.11	3.51	2.15	2.41	2.84	3.19	1.81	2.22	2.59	2.89
MC	20	10	60	1.15	.90	1.17	1.39	.69	.70	.92	1.09	.45	.55	.73	.86
	20	10	80	1.07	.84	1.10	1.32	.63	.65	.86	1.03	.41	.50	.67	.80
	20	35	60	1.45	1.17	1.45	1.67	1.00	.95	1.17	1.35	.66	.76	.94	1.08
	20	35	80	1.34	.97	1.21	1.41	.80	.76	.96	1.11	.52	.60	.76	.88
	20	60	60	2.15	1.45	1.72	1.95	1.31	1.20	1.43	1.61	.87	.98	1.16	1.31
	20	60	80	1.61	1.10	1.32	1.51	.97	.88	1.05	1.20	.42	.70	.84	.96
HDG				7.36	8.61	8.19	9.48	7.29	8.13	8.29	9.70	7.18	7.61	8.96	10.15
HDD				.79	1.01	.97	1.00	.98	.94	.92	.92	.97	.97	.88	.87

**LDV-light duty vehicles, LDT-light duty trucks, MC-motorcycles, HDG-heavy duty gas trucks, HDD-heavy duty diesel.

Table 3a. Total emission correction factor (C_T) for high altitude by calendar year, vehicle speed (mph), vehicle type (M), % hot starts (H), % cold starts (C), and temperature (T-°F).

EMISSION CORRECTION FACTORS FOR REGION: HIGH ALTITUDE

			YEAR:	1985	1985	1985	1985	1987	1987	1987	1987	1990	1990	1990	1990
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M**	H	C	T												
LDV	20	10	20	.45	.64	.73	.80	.37	.46	.52	.57	.31	.32	.36	.39
	20	10	40	.40	.58	.66	.72	.34	.42	.48	.51	.29	.30	.33	.36
	20	35	20	.84	1.13	1.29	1.42	.71	.83	.95	1.04	.61	.60	.68	.75
	20	35	40	.68	.91	1.04	1.15	.59	.65	.73	.86	.52	.53	.58	.64
	20	60	20	1.22	1.62	1.86	2.05	1.04	1.20	1.38	1.52	.90	.88	1.00	1.10
	20	60	40	.96	1.28	1.43	1.57	.85	.96	1.10	1.21	.76	.74	.84	.92
LDT	20	10	20	1.06	1.46	1.73	1.92	.81	1.17	1.37	1.52	.54	.85	.98	1.08
	20	10	40	.96	1.30	1.55	1.73	.79	1.05	1.23	1.36	.49	.74	.88	.96
	20	35	20	1.81	2.49	2.90	3.21	1.39	1.99	2.30	2.59	.91	1.45	1.67	1.84
	20	35	40	1.46	1.96	2.29	2.54	1.12	1.57	1.82	2.01	.79	1.13	1.31	1.44
	20	60	20	2.57	3.53	4.07	4.50	1.96	2.81	3.23	3.57	1.29	2.05	2.35	2.59
	20	60	40	1.96	2.67	3.03	3.35	1.50	2.09	2.41	2.66	.99	1.51	1.74	1.92
MC	20	10	20	.26	.34	.44	.52	.14	.20	.26	.30	.08	.13	.16	.19
	20	10	40	.23	.30	.39	.46	.12	.18	.23	.27	.07	.11	.14	.17
	20	35	20	.50	.63	.76	.86	.27	.37	.45	.51	.16	.24	.29	.33
	20	35	40	.38	.48	.59	.67	.21	.28	.35	.39	.12	.18	.22	.25
	20	60	20	.75	.92	1.08	1.20	.41	.55	.64	.71	.29	.35	.41	.46
	20	60	40	.54	.67	.78	.88	.29	.39	.46	.52	.17	.25	.30	.33

			YEAR:	1985	1985	1985	1985	1987	1987	1987	1987	1990	1990	1990	1990
			SPEED:	0	15	30	45	0	15	30	45	0	15	30	45
M	H	C	T												
LDV	20	10	60	.38	.54	.62	.67	.32	.40	.45	.48	.27	.28	.31	.34
	20	10	80	.36	.51	.59	.64	.30	.38	.43	.46	.26	.27	.30	.32
	20	35	60	.59	.78	.89	.98	.52	.60	.68	.75	.47	.46	.52	.57
	20	35	80	.52	.70	.80	.88	.47	.54	.62	.68	.43	.42	.48	.52
	20	60	60	.80	1.02	1.17	1.29	.72	.81	.92	1.01	.66	.64	.73	.80
	20	60	80	.69	.88	1.01	1.11	.64	.71	.81	.89	.60	.57	.65	.72
LDT	20	10	60	.89	1.21	1.45	1.62	.69	.97	1.15	1.27	.46	.70	.82	.90
	20	10	80	.85	1.14	1.39	1.55	.66	.93	1.10	1.22	.44	.67	.78	.86
	20	35	60	1.24	1.65	1.93	2.14	.95	1.32	1.53	1.70	.63	.95	1.10	1.21
	20	35	80	1.10	1.45	1.71	1.90	.85	1.17	1.36	1.51	.57	.85	.98	1.07
	20	60	60	1.58	2.08	2.40	2.66	1.22	1.66	1.92	2.12	.80	1.20	1.38	1.52
	20	60	80	1.34	1.75	2.02	2.24	1.04	1.41	1.62	1.80	.69	1.02	1.17	1.29
MC	20	10	60	.20	.27	.36	.42	.11	.16	.21	.25	.06	.10	.13	.16
	20	10	80	.18	.25	.33	.39	.10	.14	.19	.23	.05	.09	.12	.14
	20	35	60	.30	.37	.46	.53	.16	.22	.27	.31	.09	.14	.17	.20
	20	35	80	.23	.30	.37	.43	.12	.17	.22	.25	.07	.11	.14	.16
	20	60	60	.39	.48	.57	.64	.21	.28	.34	.38	.12	.18	.21	.24
	20	60	80	.28	.34	.41	.47	.15	.20	.24	.28	.08	.13	.15	.17
HDG				1.83	5.52	6.51	7.43	1.69	4.22	5.11	6.25	1.62	3.02	3.79	4.66
HDD				.06	.86	.77	.74	.09	.74	.66	.63	.03	.65	.57	.55

**LDV-light duty vehicles, LDT-light duty trucks, MC-motorcycles, HDG-heavy duty gas trucks, HDD-heavy duty diesel.

Table 3b. Total emission correction factor (C_T) for high altitude by calendar year, vehicle speed (mph), vehicle type (M), % hot starts (H), % cold starts (C), and temperature (T°F).

3. Parking Areas

a. Scope

The methodologies used to evaluate emissions from a line source could also be applied to a parking lot but such an application would require defining each parking aisle as a road segment, each crossing of aisles as an intersection, and each parking slot as a source of idling emissions. Such an application would be very time-consuming and would not provide significantly improved results over a more general method.

This section presents a general methodology for evaluating an entire parking lot as an area source. The basic assumptions are:

- All vehicles emit CO at a constant rate while operating within an area at speeds from 0 to 15 mph.
- Emissions from a parking lot are evenly distributed over the entire area of the lot.

One drawback to the general area source methodology lies in its insensitivity to high emission line sources within the area. A simple approach has been included to exclude such line sources from calculations of area source emissions and analyze the line source as in previous sections.

The parameters used to determine areawide emission rate Qa, are:

- CO emission rate per vehicle, Fs (g veh-s^{-1})
- Running time per vehicle, T(s veh^{-1})
- Vehicle hourly volume flow, V(veh hr^{-1})
- Parking lot area, A(m^2).

Both running time and volume demand can vary by direction (i.e., entrance or exit) as can the emission rate. Hence, it is necessary to define values for entrances (i.e., Te, Ve, and F) and exits (i.e., Tx, Vx, and F). The traffic parameters can also vary according to the road segment used (denoted by subscript i). Equation 15 explicitly defines the procedure that must be followed in the parking area source analysis:

$$Qa = \frac{F\Sigma_i Te_i Ve_i + F\Sigma_i Tx_i Vx_i}{3600A} \quad (gm\ m^{-2}\ sec^{-1}) \qquad (15)$$

Traffic information for individual line sources and for general parking lot features is shown in Worksheet 1. The information specifically required to perform parking lot area source analysis is shown in Table 4. Worksheet 3 presents the step-by-step procedure for computing the area source emission rate while the following sections provide the background discussion of the above parameters.

b. Emission Factor

In parking areas average vehicle speeds rarely exceed 10 to 15 mph. Because these low average operating speeds are difficult to estimate and because emissions per unit time are more applicable especially when engine idle can predominate congested conditions, an idle emission factor will be used. Hence, the base emission factor (F) for parking lots is .42 g sec^{-1} veh^{-1}. F varies with calendar year, geographic location, percentage of cold starts, vehicle mix and temperature, etc. and may also vary with direction (entrance or exit insofar as this affects the cold/hot start ratio. F, therefore, should be multiplied by the appropriate cold start/temperature correction factors from Table 1 through 3 for 0 mph for entering and exiting vehicles.

c. Traffic Demand Volume

The total traffic demand volume for a facility may be estimated by one of the techniques presented in Appendix A.

Table 4

INFORMATION USED IN PARKING LOT SOURCE ANALYSIS

Information	Remarks
Plans or blueprints of the parking lot and surroundings	Should include features such as traffic lane locations, number of lanes at gates, design of gate approaches, and design intersection approaches on access roads.
Available parking spaces	Should be divided into three elements: • Spaces available in the parking lot • Spaces in other off-street public and private parking lots that will be used to service the facility being developed. • On-street spaces available within 1 km that will be used to service the facility being developed.
Angle of parking	Affects time needed to park and unpark vehicle.
Number of parking spaces allotted for buses	Should be available from parking lot design.
Number and capacity of exit/entrance gates	
Distribution of traffic among gates	Includes information about numbers of left- and right-hand turns.
Number of buses arriving and departing during peak daily 1- and 8-hour use periods	
Stadium emptying time (if the facility being served by the parking lot is event-oriented, such as a stadium)	Time after end of an event by which all spectators have reached their parking spaces.
1- and 8-hour demand volumes for entering and exiting the parking lot	
Diurnal distribution of demand for trips to and from the parking facility	

The entering and exiting traffic demand volume per gate (Ve_i and Vx_i, respectively) should be provided by the applicant based on either: (1) the results of a comprehensive marketing or traffic study for the site, or (2) estimates provided by the developer. Volume demand is obtained by apportioning the entering and exiting traffic among the various gates and lanes, and should reflect the orientation of the gates with respect to the distribution of the user population.

d. Vehicle Running Time

The vehicle running time is a function of direction (entering or exiting) and vehicle location within the indirect source. This can be computed as the sum of two major elements or components:

- Base running time, defined as the total duration of a single vehicle's route from entrance through park and to exit, in the absence of congestion.
- Excess running time caused by delays at both congested gates or intersections, and within the parking area.

(1) Base Running Time

The entering running time for an individual vehicle is the sum of the time required to: (1) approach, (2) enter, (3) move in, and (4) park in the parking area of the indirect source. On departure the exiting running time is comprised of four similar movements: (5) unpark, (6) movement out, (7) exit, and (8) departure. During periods with little or no congestion, the total time spent in each of these eight movements defines the base running time (Brt). During peak demand periods, the actual running time may exceed the base value because of congestion and resulting delays. Normally, the base running time will be provided directly by the applicant; otherwise the Brt can be estimated from the following set of seven relationships:

$$\text{Base approach time} \quad \frac{\text{Distance between entrance and nearest intersection on the access road m}}{\text{Posted speed limit (m s}^{-1}\text{)}} \tag{16}$$

$$\text{Base entrance time} \quad \frac{1}{\text{Main entrance gate capacity (veh/s)}} \tag{17}$$

$$\text{Base movement-in time} \quad \frac{\text{Distance from center of lot to main entrance measured along traffic lanes (m)}}{\text{Speed limit in lot (m/s)}} \tag{18}$$

Base stop/start time 5-10s. Use 5 s for 45° or less angle of parking and 9 ft. (2.74 m) or greater stall width. Add 1 s for each 9° that the angle of parking exceeds 45°. Add 1 s for each 3 in. (0.08 m) that stall width is less than 9 ft. to a maximum of 10 s. (19)

$$\text{Base movement-out time} \quad \frac{\text{Distance from center of lot to main exit measured along traffic lanes (m)}}{\text{Speed limit in lot (m/s)}} \tag{20}$$

$$\text{Base exit time} \quad \frac{1}{\text{Main exit gate capacity (veh/s)}} \tag{21}$$

$$\text{Base departure time} \quad \frac{\text{Distance between main exit and nearest intersection on the access road (m)}}{\text{Posted speed limit (m/s)}} \tag{22}$$

(2) Excess Running Time

Movement in and Movement Out Time--As a rule, the components of running time most affected by congestion are the movement-in, movement-out, and entrance and exit modes. Increases in movement-in and movement out time are discussed in this section, while a subsequent section treats vehicle queueing in the entrance and exit modes.

In unsupervised parking, increases in movement-in time result from cars searching for parking spaces during periods when capacity is exceeded. The excess running time (Rmi) during these periods is a function of: (1) the average number of vehicles beyond capacity, and (2) the time required to serve each entering vehicle, where

$$Rmi = 3600 \frac{\sum_i (Ve_i - Vx_i) - (Pc - Po)}{2 \sum_i Vx_i} \qquad (23a)$$

The number of parked vehicles, Po, in a facility parking lot is the sum of the trips into the facility minus the sum of the trips out of the facility during the hours before the one being evaluated. (Where there is overnight parking, the vehicles left from previous days should be added to the total.) The parking lot capacity, Pc, is an input that should be supplied by the developer and entered in Worksheet 1. The term (Pc - Po) represents available parking spaces and should be set equal to zero if Po exceeds Pc.

If the facility being analyzed is a shopping center, the excess running time factors of Table 5 should be applied instead of Eq. (23a) when greater than 80% of the parking capacity is used.

If the parking at a facility is supervised, it is assumed that: (1) the parking lot is closed when it is filled to capacity, and (2) excess vehicles, beyond the lot capacity, are routed to auxiliary parking. If traffic is routed to auxiliary parking, an entering vehicle would have an increased running time as follows:

Table 5

GUIDELINES FOR ASSESSING THE IMPACT OF EXCEEDING PARKING CAPACITY (Pc) ON BASE RUNNING TIME AT SHOPPING CENTERS

Parking Spaces Per 1000 sq. ft. of Gross Leasable Shopping Area	Rmi (Excess Movement-In Running Time to be Added to Brt)
<4	$\frac{720\ Pc}{\Sigma Ve_i} - \frac{Brt}{2}\left(1 + \frac{\Sigma Vx_i}{\Sigma Ve_i}\right)$
$\geq 4,\ <6$	$\frac{360\ Pc}{\Sigma Ve_i} - \frac{Brt}{2}\left(1 + \frac{\Sigma Vx_i}{\Sigma Ve_i}\right)$
$\geq 6,\ \leq 8$	$\frac{180\ Pc}{\Sigma Ve_i} - \frac{Brt}{2}\left(1 + \frac{\Sigma Vx_i}{\Sigma Ve_i}\right)$
≥ 8	0 - assumes no excess parking

ΣVe_i is the sum of the entering gate volumes during the peak hour, and Pc is parking lot capacity, Brt is the base running time and ΣVx_i is the sum of the exiting gate volumes.

$$Rmi = 3600\left[\frac{\sum_{i}(Ve_i - Vx_i) - Pc + Po}{\sum_{i} Ve_i} + \frac{Laux}{Saux}\right] \tag{23b}$$

where

Laux is the distance an average vehicle will travel to the auxiliary lot (mi)

Saux is the speed at which an average vehicle will travel to the auxiliary lot (mph).

Unless there is evidence to the contrary, for conventional lots excess running times are assumed to be negligible in the unpark and movement-out modes. For lots with "stall parking" (i.e., when each vehicle does not have free access to an exit lane), however, excess running times (Rmo) should be computed. Table 6 lists typical waiting times (Rmo), as a function of the facility emptying time (Fet), before vehicles parked in such a manner can gain free access to exit lanes. Fet should be provided or estimated by the user from similar indirect sources or a comprehensive traffic study.

Table 6

Rmo--RUNNING TIMES FOR EXIT FROM PARKING STALLS

Average Cars per Stall	Rmo
1	0.000
2	0.083 Fet
3	0.138 Fet
4	0.177 Fet
5	0.210 Fet
6	0.235 Fet
7	0.235 Fet

Running Time at an Exit or Entrance--Increased running times occur in the exit or entrance modes when volume demand approaches gate capacity. When this happens, queues form at the exits (entrances) thereby increasing running times. The increase in running time at a gate with no traffic signal may be estimated from queueing theory using the following generalized equation:

$$Rq_i = \frac{3600\ V_i}{C_i(C_i - V_i)} \tag{24}$$

where

Rq_i = time (s) spent in queue at each approach i

C_i = entrance or exit capacity of approach i of the gate (vph).

The excess running time in entering (Re) or exiting (Rx) the facility is also a function of the appropriate demand volume-to-capacity ratio as follows:

$$Re_i = \frac{3600\ Ve_i}{Ce_i(Ce_i - Ve_i)} \qquad Ve_i/Ce_i \leq 0.95 \tag{25a}$$

$$Re_i = \frac{7.2\text{x}10^4}{Ce_i} \qquad 0.95 < Ve_i/Ce_i \leq 1 \tag{25b}$$

$$Re_i = \frac{7.2\text{x}10^4}{Ce_i} + \frac{Ve-Ce_i}{2\ Ce_i} \qquad Ve_i/Ce_i > 1 \tag{25c}$$

where

Re_i is the excess running time at entrance gate i to the facility

Ce_i is the entrance capacity of entrance gate i

Ve_i is the volume demand for entrance gate i.

$$Rx_i = \frac{3600\ Vx_i}{Cx_i\ (Cx_i - Vx_i)} \qquad Vx_i/Cx_i \leq 0.95 \qquad (26a)$$

$$Rx_i = \frac{7.2x10^4}{Cx_i} \qquad 0.95 < Vx_i/Cx_i \leq 1 \qquad (26b)$$

$$Rx_i = \frac{7.2x10^4}{Cx_i} + \frac{Vx_i - Cx_i}{2\ Cx_i} \qquad Vx_i/Cx_i > 1 \qquad (26c)$$

where

Rx_i is the excess running time at exit gate i to the facility

Cx_i is the capacity of exit gate i to the facility

Vx_i is the volume demand for exit gate i.

The general methodology above applies to vehicles exiting shopping center parking lots and other similar facilities for which approximately uniform hourly departure rates have been predicted. On the other hand, a stadium, arena, or business often has a peak demand, which lasts for less than one hour. Another method of determining excess running time, Rx, when leaving a facility with heavily peaked demand is to use an estimate based on the facility emptying time (Fet) and the parking lot emptying time (Plet). Fet is the number of seconds that it takes the occupants of a facility to reach their vehicles. Plet, on the other hand is the time it takes all parked vehicles to exit the facility parking lot. Both Fet and Plet should be provided by the applicant. Fet is subtracted from Plet to determine an average vehicle excess running time as follows:

$$Rx_i = \frac{1800\ Vx_i}{Cx_i} - \frac{Fet}{2} \qquad (27)$$

where

$\frac{1800\ Vx_i}{Cx_i}$ is Plet in seconds.

$\frac{Fet}{2}$ is the average time a person will arrive at his vehicle after the end of an event, in seconds.

(3) Total Vehicle Running Time

The following equation should be used to estimate running time for a typical vehicle-trip entering a parking lot during the one- or eight-hour period of interest.

$$Te_i = \frac{Brt}{2} + Rmi + Re_i \qquad (28)$$

where

Te_i is the total running time for an entering vehicle.

Brt is the base running time and is the sum of equations 16 through 22.

Rmi is the excess running time due to movement-in and is found from equation 23a, 23b or Table 5.

Re_i is the excess running time due to queueing at an entrance gate and is found from equation 25a, 25b or 25c.

It is necessary to divide the base running time by 2 in the equations above since each trip is a one-way trip and Brt is the time to both enter and leave the parking lot. In determining average daily trip generation rate, each vehicle is counted twice. If the lot is supervised, Rmi reflects the extra movement-in time.

The total exiting running time for a typical vehicle trip from a parking lot for the one- or eight-hour period of interest is

$$Tx_i = \frac{Brt}{2} + Rmo + Rx_i \qquad (29)$$

where

Tx_i is the total running time for an exiting vehicle

Brt is the base running time and is the sum of equations 16 through 22.

Rmo is the excess running time due to movement out and is found from Table 6 for lots with stall parking or assumed equal to zero for others.

Rx_i is the excess running time due to queueing at an exit and is found from equation 26a, 26b, 26c or 27.

It should be noted that if it takes longer than an hour to empty the parking lot, the above equations will overestimate Rx for a one-hour period.

e. Line Source Emissions within Area Sources

So far the area source methodology computes emissions and assumes them to be evenly distributed over a parking area. Average running times for exiting (Tx) vehicles have been determined for each exit approach i. In computing Tx_i, a queued running time Rx_i was added, as appropriate. This element of running time should be subtracted from the Tx_i value when road segment i is to be considered independently as a line source.

There are two limitations in computing emissions from a line source within an area source that should be mentioned. First, the dispersion model assumes a line source is a straight line. In many parking lots this may not be true; the queue often consists of a main trunk, with many feeders branching into it. Second, the approaches to exit or entrance gates are often very short and a vehicle queue may extend beyond the length of the approach. When either situation occurs the methodology presented here is inadequate, and a more detailed analysis should be undertaken by the user. This should include a detailed study of the queueing exit and entrance gates, number and lengths of internal line sources, and volume demand and capacities. Possibly all feeder roadways and vehicles extending beyond the approaches could be separately treated as short line sources.

To determine the effects of a line source located within an area source the following guidance is provided. (Note that line sources within the area source that may develop a vehicle queue are always exit links. No method is outlined in the area source review for eliminating input-link running times from area source emissions, so that such links may be analyzed as line sources).

(1) Identify any exit approach (i) that is to be considered as a separate line source. If it meets the two limitations mentioned above, proceed. If not, do not consider the line source separately.

(2) Calculate the emission contribution of each exit approach identified in Step 1.

$$Qal_i = \frac{F\ Rx_i\ Vx_i}{3600\ A} \qquad (30)$$

Qal_i - emission contribution of an exit approach to the total area emission rate (gm/m^2 sec)

Other symbols are the same as used previously.

(3) Subtract all line source emissions from the area source emission rate.

$$Qa' = Qa - \Sigma Qal_i \qquad (31)$$

Qa' - area emission rate without exit approach contributions. Use Qa to complete subsequent area source dispersion estimates.

(4) Complete Worksheet 2 for each exit subtracted from the area emissions in Step 3 (see next step).

(5) The following equations supercede steps 8 and 9 respectively on Worksheet 2:

a. Determine the length of the queue along the line source, i.e.,

$$Lq_1 = \frac{8}{M}\frac{Vx_1Cy}{3600} + \frac{Rx_1Cx_1}{3600} \quad \text{for a signalized intersection} \qquad (32a)$$

$$Lq_1 = \frac{8}{M}\frac{Rx_1Cx_1}{3600} \quad \text{for a nosignalized intersection} \qquad (32b)$$

b. Determine the average excess running time on the line source, i.e.,

$$Rq_1 = \frac{Cy - G}{2} + Rx_1 \quad \text{for a signalized intersection}$$

$$Rq_1 = Rx_1 \quad \text{for a nonsignalized intersection}$$

(6) Follow the usual procedures outlined in this chapter to determine the area source and line source impacts on a receptor using Worksheets 4, 5 and 6.

Worksheet 1

TRAFFIC INFORMATION USED IN THE APPLICATION OF THE EVALUATION PROCEDURE

1. Road segment or intersection approach identification

2. Observed 1-hr volume (vph)

Observed 8-hr volume (vph)

Projected 1-hr peak demand (vph)

Projected 8-hr peak demand (vph)

3. Percentage cold starts

4. Percentage trucks and buses

5. Metropolitan population

6. Slope

7. Free-flow parameters
   - Number of lanes
   - Average lane width (ft)
   - Design speed (mph)
   - Highway type (see Figures 2-5)

8. Intersection parameters
   - Intersection designation
   - Approach width (ft)
   - Percentage right turns
   - Percentage left turns
   - Type control and description of signal controller

9. Area source parameters
   - Parking lot gate designation
   - Projected 1-hr peak entrance demand (vph)
   - Projected 1-hr peak exit demand (vph)
   - Projected 8-hr peak entrance demand (vph)
   - Projected 8-hr peak exit demand (vph)

   - Parking lot area (m^2)
   - Parking lot capacity (veh)
   - Running time required to access auxiliary parking (s)
   - Facility emptying time
   - Average cars per stall
   - Average area per stall (m^2)

INSTRUCTIONS FOR COMPLETING WORKSHEET 1

Step	Instructions
1	Number each road segment or intersection approach. All entrance and exit segments at a facility should be numbered as well as all intersection approaches at major intersections within 1/4 mile (0.4 km) of the facility boundary. A separate number should be assigned to each direction of roadway traffic flow.
2	Enter representative historical traffic volumes for all segments identified in Step 1. Assign projected volumes to all segments on the basis of an origin-destination study or other methods (see Appendix A). Use diurnal traffic patterns to make projections for the peak hours of the day.
3	Enter estimates of cold starting vehicles on each road segment in percentage. (User or developer supplies or see Midurski, et. al., 1977).
4	Enter an estimate of the percentage of trucks and buses in the traffic stream. The effects of trucks and buses in estimating indirect source impact is not negligible, however, their effects are minimal during peak hours since heavy trucks usually avoid rush hours and buses are diesel (minimal CO emissions). The percentage is also used in the capacity analysis of Appendix B to adjust the service capacities. Trucks and buses may be included in the emission factors. (User or developer supplies.)
5	Enter the approximate metropolitan population.
6	Enter the approximate slope of the highway, intersection, or area source. This can be estimated from highway engineering practices or field measurements. It is not used at this time in the calculations.
7	Enter all dimensions and engineering characteristics of each free-flow road segment. These should be provided by the developer or estimated by the user based upon previous and similar studies or upon a comprehensive planning and traffic study.
8	Enter the traffic and road characteristics of each intersection. Each intersection approach width should include turn channels, and the signal control information should identify the phases which control each turning movement (it is assumed that turn channel lengths are adequate to store peak demand for turning movements). All information should be provided by the developer or estimated by the user based upon previous and similar studies or upon a comprehensive planning and traffic study.
9	Parking lot characteristics should be provided by the developer including volumes, times and dimensions. The user may make estimates based upon previous and similar studies if applicable to the area of interest.

WORKSHEET 2--LINE SOURCE EMISSION RATE COMPUTATION
(see instructions following)

Project No.:________________ Analyst: ________________

Site: ________________ Date:________________

Step	Symbol	Input/Units	Traffic Stream
1	i	Road segment (or approach identification)	
2	V_i	Demand volume (vph)	
3	C_i	Free-flow capacity (vph)	
4	S_i	Cruise speed (mph)	
5	Ef_i	Free-flow emissions (g/veh-m)	
6.1	M_i	Number of lanes in approach i	
6.2	j	Signalized intersections phase identification	
6.3	$Cs_{i,j}$	Capacity service volume of approach i for phase j (vph of green)	
6.4	$V_{i,j}$	Demand volume for approach i, phase j (vph)	
6.5	C_y	Signal cycle length (s)	
6.6	$G_{i,j}$	Green phase length for approach i, phase j (s)	
6.7	C_i	Capacity of approach i (vph)	
6.8	$P_{i,j}$	Proportion of vehicles that stop	
6.9	$N_{i,j}$	Number of vehicles that stop per signal cycle	
7	N_i	Average number of vehicles in queue at four way stop or two-way stop or end of green phase	
8	Lq_i	Length of vehicle queue for approach i (veh-m/lane)	
9	Rq_i	Average excess running time on approach (s/veh)	
10	Ea_i	Excess emissions from acceleration (g/veh-m)	
11	Ed_i	Excess emissions from deceleration (g/veh-m)	
12	Qad_i	Excess emission rate from acceleration and deceleration (g/m-s)	
13	Lad_i	Length of acceleration and deceleration (m)	
14	Le_i	Length over which excess emissions apply (m)	
15	Fs_i	Average idling emission rate (g/s)	
16	Qe	Average excess emission rate (g/m-s)	
17	Qe'_i	Adjusted excess emission rate (g/s-m)	
18	Qfc_i	Free-flow emission rate (g/s-m)	

INSTRUCTIONS FOR COMPLETING WORKSHEET 2

Step	Instructions
1	Assign a unique number to each road segment (or intersection approach) that is of prime consideration.
2	Enter demand volume from Worksheet 1.
3	Enter free-flow capacity from Worksheet B, line 2.4 (Appendix B).
4	Divide line 2 by line 3 (V/C) and use the applicable figure: 2, 3, 4, or 5 to determine cruise speed. When using Figure 5 for a design speed not on Curves I, II or III, interpolate a cruise speed as in example on p. 17.
5	Enter "free flow" emission using cruise speed from line 4 to enter Figure 6.
6.1	Enter the number of lanes of each approach.
6.2	Assign a number to each signal green-phase and enter this number under each approach that moves on the green-phase.
6.3	Enter the capacity service volume of the intersection approach for this phase from Worksheet B, line 3.6.
6.4	Enter demand volume from line 4.1 Worksheet B.
6.5	Enter a signal cycle time from line 4.5 Worksheet B.
6.6	Enter a green-phase length from line 4.6 Worksheet B.
6.7	Enter approach capacity from line 7 Worksheet B.
6.8	Divide 1.0 minus (line 6.6 divided by line 6.5) by 1.0 minus (line 6.4 divided by line 6.3) and enter this as the proportion of vehicles that stop.
6.9	Multiply line 6.8 by line 6.4 by line 6.5 and divide by 3600. Enter this as the number of vehicles that stop per signal cycle.
7	Divide line 2 by the difference between 6.7 and line 2. Enter this as the average number of vehicles in a queue. If line 2 exceeds line 6.7, then an overcapacity demand exists and the user needs a more in-depth analysis than provided by this review procedure.

Step	Instructions
8	Total the stopped vehicles at the end of the red-phase from line 6.9, for approach i, and for each green phase j, to that approach. Add the average queue (line 7) multiply the sum by 8, and divide by the total lanes in each approach (line 6.1) to obtain the length of the vehicle queue.
9	For signalized intersections, multiply the difference between line 6.5 and line 6.6 by line 6.8 and line 6.4, sum this result for all phases controlling the approach, and divide by 2.0 times line 2. Add to this, line 7 multiplied by 3600 divided by line 6.7. For unsignalized intersections multiply line 7 by 3600 and divide by line 6.7. Enter this as the average excess running time on the approach.
10	Enter excess emissions Ea, using line 4 to enter Figure 7.
11	Enter excess emissions Ed, using line 4 to enter Figure 7.
12	Multiply the sum of lines 10 and 11 by the sum over each approach of line 6.9 and divide by line 6.5. Enter this as the excess emission rate from acceleration and deceleration.
13	Square line 4 and multiply by 0.0894. Enter this as the length through which acceleration and deceleration take place.
14	Enter the larger length of vehicle queue: line 8 or 40 m.
15	Subtract line 4 divided by 5.0 from line 9. Multiply the result by 0.42 and by line 2, and divide by 3600. Enter this as the average idling emissions rate.
16	Add line 15 to the product of line 12 times line 13, and divide the result by line 14. Enter this as the average excess emission rate.
17	Find the total emission correction factor (C_T) for the desired vehicle mix, cold start %, hot start %, temperature, calendar year location, altitude, and idle mode (0 mph).* $C_T = LDV(CF) + LDT(CF) + HDG(CF) + HDD(CF) + MC(CF)$

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*Catalyst and non-catalyst vehicles are calculated into the tables.

Step Instructions

where

LDV - fraction of vehicle mix that is light duty vehicles

LDT - fraction of vehicle mix that is light duty trucks, including class I (<6000 lbs GVW) and class II (≥6000-8500 lbs GVW)

HDG - fraction of vehicle mix that is heavy duty gas vehicles

HDD - fraction of vehicle mix that is heavy duty diesel vehicles

MC - fraction of vehicle mix that is motorcycles

CF - correction factor per vehicle class from Table 1, 2 or 3.

Vehicle mix is specified by the user but a suggested vehicle mix near indirect sources in peak hour traffic is .88, .08, .03, .01, 0.

Multiply this C_T times line 16 and enter as the adjusted average excess emission rate.

18 Find another total emission factor as in line 17 but for the cruise speed in line 4. If each approach has a different cruise speed form different $C_{T's}$ may result.

Multiply line 5 by line 2, divide by 3600, and multiply by the C_T for each approach. Enter this as the adjusted free-flow emission rate.

WORKSHEET 3--AREA SOURCE EMISSIONS COMPUTATION
(see instructions following)

Project No.: ____________________ Analyst: ____________________

Site: ____________________ Date: ____________________

Step	Symbol	Input/Units				
1	Brt	Base running time	____	____	____	____
1.1		Base approach time(s)	____			
1.2		Base entrance time(s)	____			
1.3		Base movement-in time(s)	____			
1.4		Base stop, base start time(s)	____			
1.5		Base movement-out time(s)	____			
1.6		Base exit time(s)	____			
1.7		Base departure time(s)	____			
1.8		Total base running time(s)	____			
2	A	Area of parking lot (m^2)	____			
3	i	Entrance approach identification	____	____	____	____
4	Ve_i	Entrance demand volume (vph)	____	____	____	____
5	Ce_i	Entrance approach capacities (vph)	____	____	____	____
6	i	Exit approach identification	____	____	____	____
7	Vx_i	Exit demand volume (vph)	____	____	____	____
8	Cx_i	Exit approach capacities (vph)	____	____	____	____
9		Number of parking spaces occupied	____			
10	F	Emissions	____	____	____	____
11	Pc	Capacity of parking lot (veh)	____			
12	Rmi	Excess movement-in time(s)	____			
13	Fet	Facility emptying time(s)	____			
14		Excess running time	____			
14.1	Ve_i/Ce_i	Entering volume-to-capacity ratio	____	____	____	____
14.2	Vx_i/Cx_i	Exiting volume-to-capacity ratio	____	____	____	____
14.3	Re_i	Excess running time entering parking lot	____	____	____	____
14.4	Rx_i	Excess running time exiting parking lot	____	____	____	____
15	Te_i	Total entering running time (s/veh)	____	____	____	____
16	Rmo	Excess running time moving out of parking stalls (s/veh)	____			
17	Tx_i	Total exiting running time (s/veh)	____	____	____	____
18	Qa	Total emission rate from a parking lot ($g/m^2 - s$)	____			
19	Qa'	Area source emission rate without the emissions from internal road segment, i	____			

INSTRUCTIONS FOR COMPLETING WORKSHEET 3

Step	Instructions
1.1	Enter estimate of base approach time (see Eq. [16]).
1.2	Enter base entrance time (see Eq. [17]).
1.3	Enter base movement in time (see Eq. [18]).
1.4	Enter base stop/start time (see Eq. [19]).
1.5	Enter base movement out time (see Eq. [20]).
1.6	Enter base exit time (see (Eq. [21]).
1.7	Enter base departure time (see Eq. [22]).
1.8	Sum lines 1.1 through 1.7 to obtain the total base running time.
2	Enter the area of the parking lot from Worksheet 1.
3	Enter entrance approach identification i, from Worksheet 1.
4	Enter entrance demand volumes from Worksheet 1.
5	Enter entrance approach capacity: sum of (line 6.7* (leg opposite entrance) times % through traffic going to approach i) plus (line 6.7* (leg to the left of entrance, see Figure B-4) times % right turners going to approach i) plus (line 6.7* (leg to the right of entrance, see Figure B-4) times % left turners going to approach i) divided by 100.
6	Enter approach identification i, from Worksheet 1.
7	Enter exit demand volumes from Worksheet 1.
8	Enter exit capacity from line 6.7* (leg of intersection that exits the indirect source).
9	Enter the number of parking spaces occupied at the beginning of the time period of interest.
10	Multiply .42 (the average emissions for a slow moving vehicle for 1977 at low altitude, gm s^{-1} veh^{-1}) by the total emissions correction factor, C_T. Use ambient temperature, percent cold start, percent hot start, calendar year, speed, location, altitude, and vehicle mix: $C_T = LDV(CF) + LDT(CF) + HDG(CF) + HDD(CF) + MC(CF)$ Parameters are defined in Worksheet 2 instructions. Enter the result of .42 C_T as the emissions in line 10.
11	Enter the capacity of the parking lot from Worksheet 1.

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*See Worksheet 2 for appropriate intersection/gate for this entrance/exit.

Step	Instructions
12	If the facility is a shopping center and parking exceeds 80% of capacity, enter Rmi from Table 5. If the parking lot does not serve a shopping center and is unsupervised, take the sum over all i's of line 4 minus line 7 and then subtract line 11 and add line 9. Finally divide by the sum over all i's of line 7 and multiply by 1800. If the parking lot is supervised, take the sum over all 1's of line 4 minus line 7 and then subtract line 11 and add line 9. Finally divide by the sum over all i's of line 4 and multiply by the time (in seconds) it takes to access the auxiliary parking lot. (Do not enter a value that is less than zero.)
13	Enter a facility emptying time from Worksheet 1.
14.1	Divide line 4 by line 5 to obtain the entering volume to capacity ratio.
14.2	Divide line 7 by line 8 to obtain the exiting volume to capacity ratio.
14.3	If line 14.1 is less than 0.95, divide line 4 by the difference between lines 5 and 4, and multiply the result by 3600 divided by line 5. If line 14.1 is greater than 0.95 and less than 1, divide 72,000 by line 5. If line 14.1 is greater than or equal to 1, divide 72,000 by line 5, and add the product of 1800 times the difference between line 4 and line 5 divided by line 5. This gives the excess running time for entering the parking lot.
14.4	If the facility is a stadium, enter line 7 times 1800 divided by line 8, and subtract line 13 divided by 2. Otherwise, if line 14.2 is less than 0.95, divide line 7 by the difference between line 8 and line 7 and multiply the result by 3600 divided by line 8. If line 14.2 is greater than 0.95 and less than 1, divide 72,000 by line 8. If line 14.2 is greater than or equal to 1, divide 72,000 by line 8, and add the product of 1800 and the difference between line 7 and line 8 divided by line 8. This gives the excess running time for exiting the parking lot.
15	Divide line 1.8 by 2 and add line 12 plus line 14.3 to obtain the total entering running time.
16	Use line 13, and average cars per stall to enter Table 6 and determine Rmo.
17	Divide line 1.8 by 2 and add line 16 plus line 14.4 to obtain the total exiting running time.

Step	Instructions
18	For each entrance, find the product of lines 15, 4, and 10, and for each exit find the product of lines 17, 7, and 10. Sum the products for all entrances and exits and divide the result by the product of (3600 and line 2). This gives the total emission rate.
19	Subtract the product of (lines 10, 17, and 7) divided by (3600 times line 2) from line 18 for each road segment that is to be analyzed as a line source. This, then, is the area source emission rate minus the emissions from internal road segments.

D. Determination of Local Hourly CO Concentrations

This section presents methods for calculating CO concentrations for various types of vehicular line and area sources. Generalized meteorological and terrain factors are considered in terms of their influence on CO dispersion. The dispersion computations are made using CO emission rates, determined in the preceding section, and graphical dispersion methods developed from the HIWAY model (Zimmerman, et al. 1975) for line sources and from the APRAC model (Ludwig, et al. 1972) for area sources. Only one-hour average CO concentrations from each source are considered in this section. These estimates are the basis for Section E which presents methods for estimating total CO concentrations (including background) for both 1-hour and 8-hour averaging times.

1. Atmospheric Stability and Surface Roughness

Atmospheric stability and surface roughness are two important parameters in the dispersion of CO from mobile sources. Atmospheric stability is important in that it helps characterize the mixing potential of the atmosphere, while surface roughness indicates the initial ground level turbulence into which the exhaust plume will be released. These, coupled with wind speed and direction, are the controlling meteorological variables used in this dispersion analysis.

Because the purpose of these guidelines is to estimate whether CO concentrations will exceed the ambient air quality standard, all stabilities need not be used. Only those stabilities that are most likely to result in high concentrations will be considered. In this case these stabilities are D, E, and F (neutral to stable) as classified in Turner's Workbook (1970).

To treat both the stability and the mixing effects of urban development properly, the dispersion nomograms presented later allow the user to specify conditions appropriate to his site. Most conditions are summarized in Table 7. To use the table, first select the appropriate category; next, select the appropriate stability and initial dispersion (σ_{z_0}) on the basis of the land use type (and/or the surface roughness which is related to height and number of buildings or other obstacles, e.g. urban areas would be classified as rough surfaces). [More complete information concerning the determination of stability classes can be found in Turner (1970) and Ludwig and Dabberdt (1975)].

Table 7

STABILITY CLASSIFICATION

Condition	Category
Day	I
Night, wind ≥ 3 m s^{-1}	I
Night, wind ≥ 2 m s^{-1} and cloud cover $\geq 5/10$	I
Night, wind < 2 m s^{-1}	II
Night, wind < 3 m s^{-1} and cloud cover $< 5/10$	II

Category	Rural		Suburban		Urban	
	SC	σ_{z_o} (m)	SC	σ_{z_o} (m)	SC	σ_{z_o} (m)
I	E	1.5	D	5.0 (1.5)*	D	5.0
II	F	1.5	E	5.0 (1.5)*	E	5.0

*A σ_{z_o} of 1.5 m is to be used in suburban areas if the source is not closer than 10 times the building height to any building.

2. Computation of CO Concentrations

The dispersion analysis requires three categories of input data:

- Emissions
- Meteorology
- Receptor location.

The output is the ambient CO concentration attributable to local sources. The total CO level is obtained by adding the background concentration (this is demonstrated in Section E of this chapter). The dispersion analysis is done for one or more of three types of local sources:

- Continuous line sources with uniform emissions
- Discrete line sources with varying emissions
- Distributed or area sources with uniform emissions.

Computation of the CO contribution from each of these sources is given in the three following sections.

a. Continuous Line Source

A continuous (i.e., effectively infinite) line source is considered here to be a section of highway on which the emission rate is both uniform and continuous for at least a specified or minimum length of roadway. This minimum roadway length is a function of atmospheric stability, initial dispersion (σ_{z_o}), wind/roadway angle, and road/receptor separation. Nonuniformity of emissions beyond the minimum length will not affect the concentrations more than approximately 2% at the specified receptor location. Table 8 lists minimum roadway lengths as determined by the application of the HIWAY dispersion model for roadway lengths up to 3860 (Zimmerman and Thompson, 1975).

Table 8

MINIMUM ROADWAY LENGTH* (m) OF AN "INFINITE" LINE SOURCE

Stability Class	σ_{z_0} (m)	Road/Receptor Separation (m)	Wind/Roadway Angle (deg)							
			0	9	18	27	41	53	71	90
D	1.5	10	>3920	1040	320	160	120	120	100	100
D	1.5	15	>3920	3920	320	160	120	120	100	100
D	1.5	20	>3920	3920	320	200	140	120	100	100
D	1.5	40	>3920	3920	320	320	160	140	120	100
D	1.5	80	>3920	3920	560	320	320	200	140	100
D	1.5	160	>3920	>3920	1040	560	320	320	160	120
E	1.5	10	>3920	560	200	140	120	100	100	100
E	1.5	15	>3920	1040	200	140	120	120	100	100
E	1.5	20	>3920	1040	320	160	120	120	100	100
E	1.5	40	>3920	1040	320	320	160	140	120	100
E	1.5	80	>3920	>3920	560	320	200	160	140	100
E	1.5	160	>3920	>3920	1040	560	320	320	140	120
F	1.5	10	>3920	320	160	120	120	100	100	100
F	1.5	15	>3920	560	200	140	120	100	100	100
F	1.5	20	>3920	560	200	160	120	120	100	100
F	1.5	40	>3920	1040	320	200	160	120	120	100
F	1.5	80	>3920	1040	560	320	200	160	120	100
F	1.5	160	>3920	>3920	1040	560	320	320	160	100
D	5.0	10	>3920	560	200	140	120	100	100	100
D	5.0	15	>3920	3920	320	160	120	120	100	100
D	5.0	20	>3920	3920	320	200	140	120	100	100
D	5.0	40	>3920	3920	320	200	160	140	120	100
D	5.0	80	>3920	>3920	560	560	320	200	140	100
D	5.0	160	>3920	>3920	1040	560	560	320	160	120
E	5.0	10	>3920	560	200	140	120	100	100	100
E	5.0	15	>3920	560	200	140	120	120	100	100
E	5.0	20	>3920	1040	320	160	120	120	100	100
E	5.0	40	>3920	1040	320	200	160	140	120	100
E	5.0	80	>3920	>3920	560	320	200	160	140	100
E	5.0	160	>3920	>3920	1040	560	320	320	160	120

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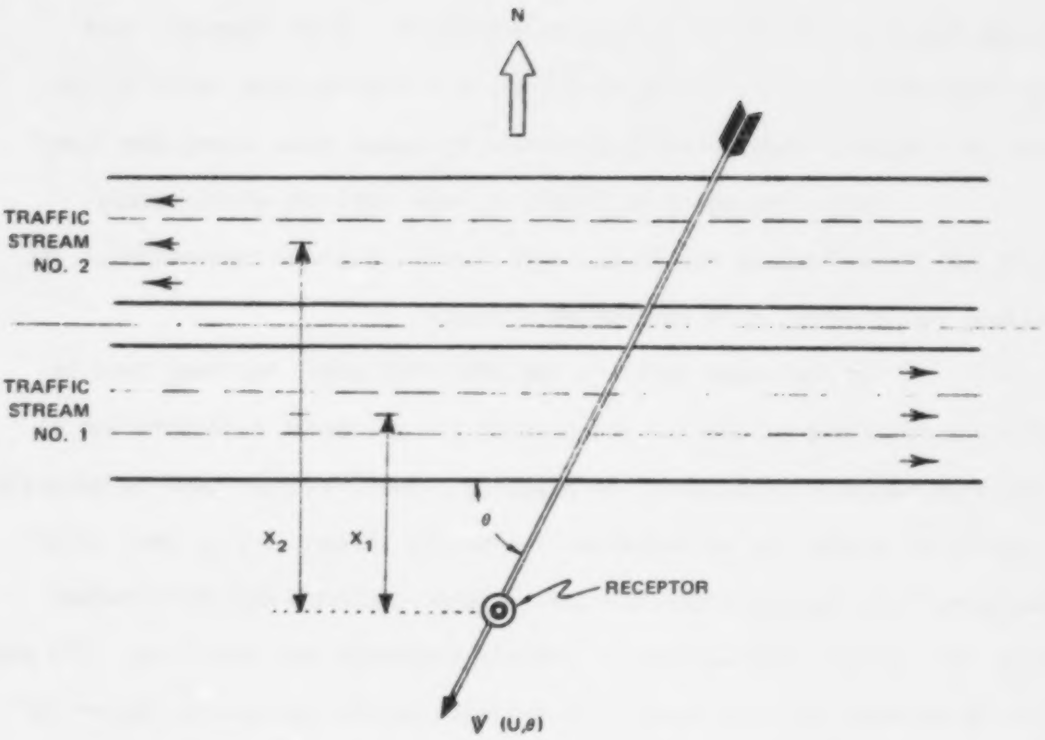
[*] Minimum roadway length includes a correction factor for overlap of wind sectors.

The HIWAY dispersion model is not appropriate for those roadway configurations in which local wind circulations dominate. For example, urban street canyons are characterized by local effects that can cause the pollutant concentration on the upwind (or leeward with respect to the rooftop wind direction) side of the street to be significantly larger than on the downwind side (or windward) (Johnson et al., 1971; Ludwig and Dabberdt, 1972). Appendix C gives a short technique to check for street canyon effects and is a description of a street canyon model that can be applied in these special cases. (Using Appendix C would supercede carrying out steps 8 through 12 on Worksheets 4 and 5).

The geometry and nomenclature of a continuous roadway is illustrated in Figure 8. Note that the road/receptor separation (X) is specified as the perpendicular distance from the center of a given traffic stream to the receptor. In the same way, emissions from all of the traffic lanes can be considered to be emitted from the central lane of the traffic stream. A comparison has been made (Dabberdt and Sandys, 1976) of the concentration predicted at several near-roadway receptors by each of two methods: (1) considering separately emissions and dispersion from each of three adjacent traffic lanes having identical emission rates, and (2) considering all emissions to originate from the central lane only. Comparisons among cases of several wind/roadway angles and differing stability showed virtually no differences in the concentrations


N
TRAFFIC STREAM NO. 2
TRAFFIC STREAM NO. 1
X_2
X_1
θ
RECEPTOR
$\mathbb{V}$ (U,θ)

X_1 = Distance from center traffic stream 1 to receptor
X_2 = Distance from center traffic stream 2 to receptor
$\mathbb{V}$ = Wind vector
θ = Wind-roadway angle
U = Vector wind speed

FIGURE 8. "INFINITE" ROADWAY GEOMETRY

predicted by the two methods. The reasons for advocating the use of an "effective" central lane are: (1) differences in emission rates among lanes cannot usually be resolved, and (2) the added brevity and reduced source for error of the CO estimation procedure. Note, however, that the dispersion analysis can be performed on a lane-by-lane basis if the user can identify significantly different emission rates among the lanes.

Referring again to Figure 8, note that the wind/roadway angle (θ) is defined as the acute angle formed by the intersection of the roadway longitudinal axis and the wind vector.

As indicated earlier, the EPA HIWAY model has been used to estimate pollutant dispersion and predict the pollutant concentration at specified receptor locations. In practice, HIWAY has been used to generate a family of graphs for an infinite line source (Figure 9 a-e) that relate the normalized concentration to road/receptor distance and wind/roadway angle for various combinations of terrain roughness and stability. (It should also be pointed out that unless the optional height correction factor is applied from Figure 9 or 10, concentrations are calculated at 1.8 m above the ground). The normalized concentration is defined as the product of the pollutant concentration (χ, g m^{-3}) and vector wind speed (U, m s^{-1}) divided by the emission rate (Q, g m^{-1} s^{-1}). The pollutant concentration is obtained by multiplying the normalized concentration by Q and dividing by U.

The dispersion analysis for an infinite line source is summarized in Worksheet 4. This procedure estimates the one-hourly CO impact of the source under analysis. To this value must be added the CO contribution from: (1) other nearby line sources--both infinite and finite, (2) nearby

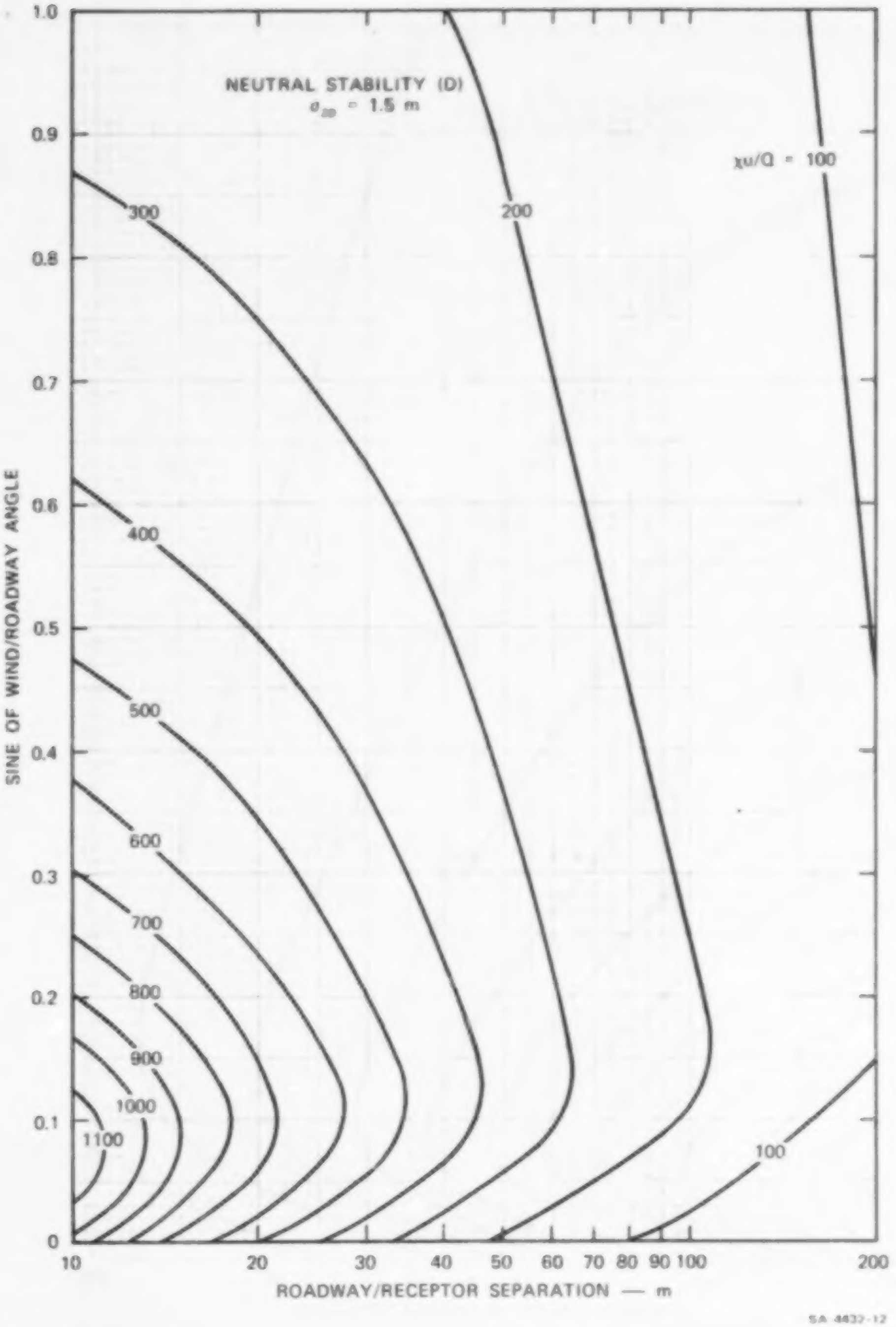


FIGURE 9a VALUES OF $\chi u/Q$ ($10^{-3}m^{-1}$) FOR VARIOUS ROADWAY/RECEPTOR SEPARATIONS AND WIND/ROADWAY ANGLES; INFINITE LINE SOURCE

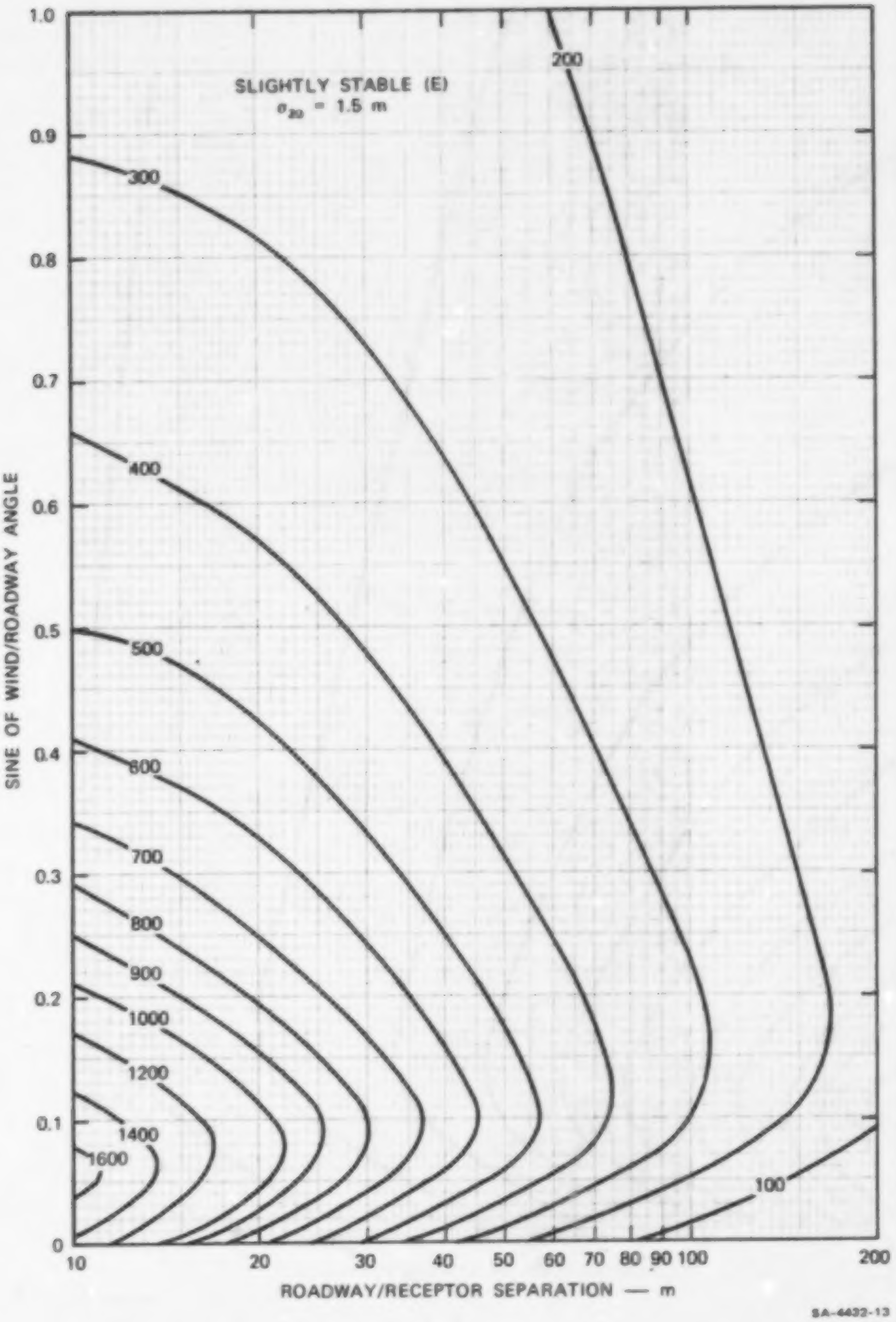


FIGURE 9b VALUES OF $\chi u/Q$ ($10^{-3}m^{-1}$) FOR VARIOUS ROADWAY/RECEPTOR SEPARATIONS AND WIND/ROADWAY ANGLES; INFINITE LINE SOURCE

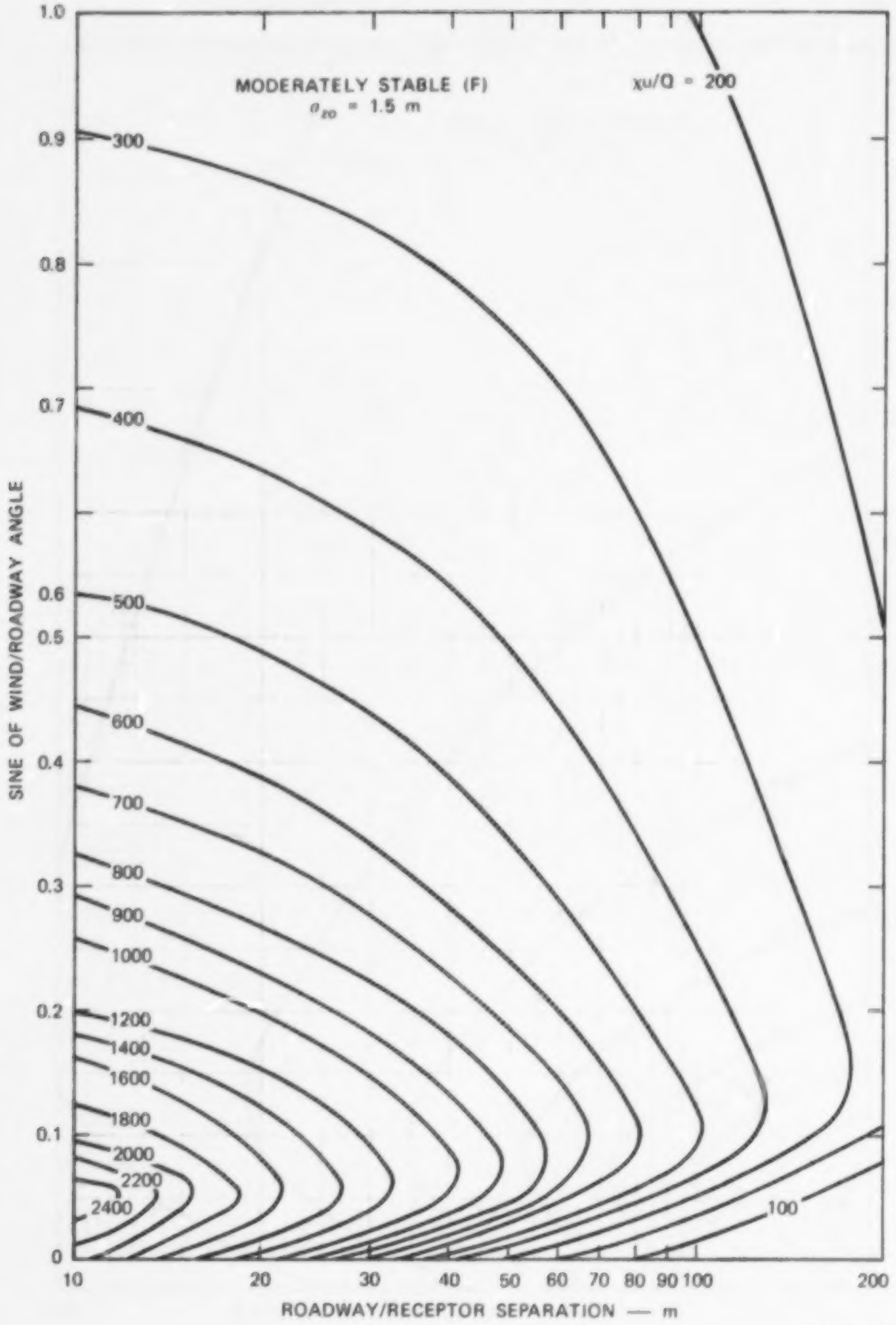


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FIGURE 9c VALUES OF $\chi u/Q$ ($10^{-3} m^{-1}$) FOR VARIOUS ROADWAY/RECEPTOR SEPARATIONS AND WIND/ROADWAY ANGLES; INFINITE LINE SOURCE

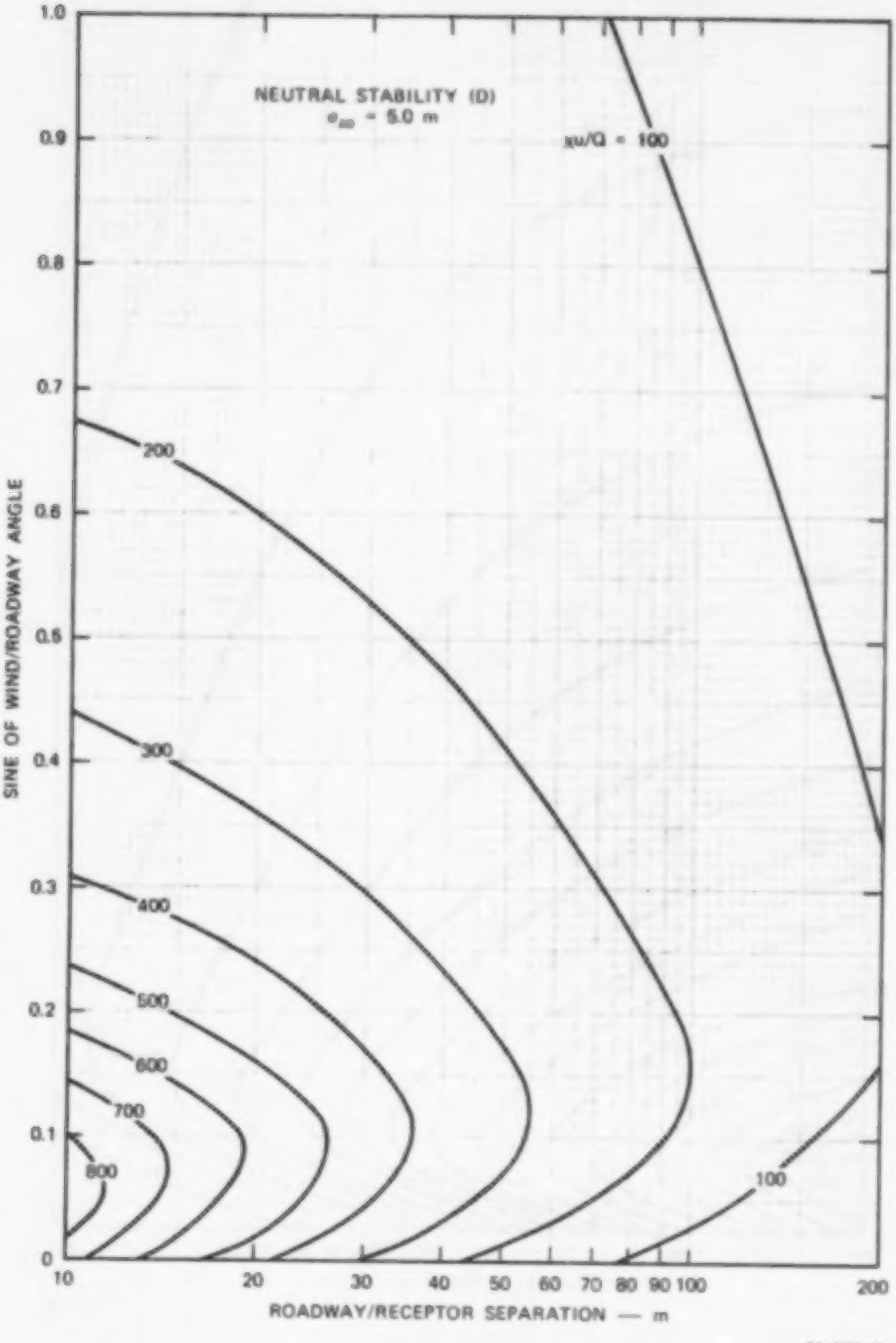

FIGURE 9d VALUES OF $\chi u/Q$ ($10^{-3}m^{-1}$) FOR VARIOUS ROADWAY/RECEPTOR SEPARATIONS AND WIND/ROADWAY ANGLES; INFINITE LINE SOURCE

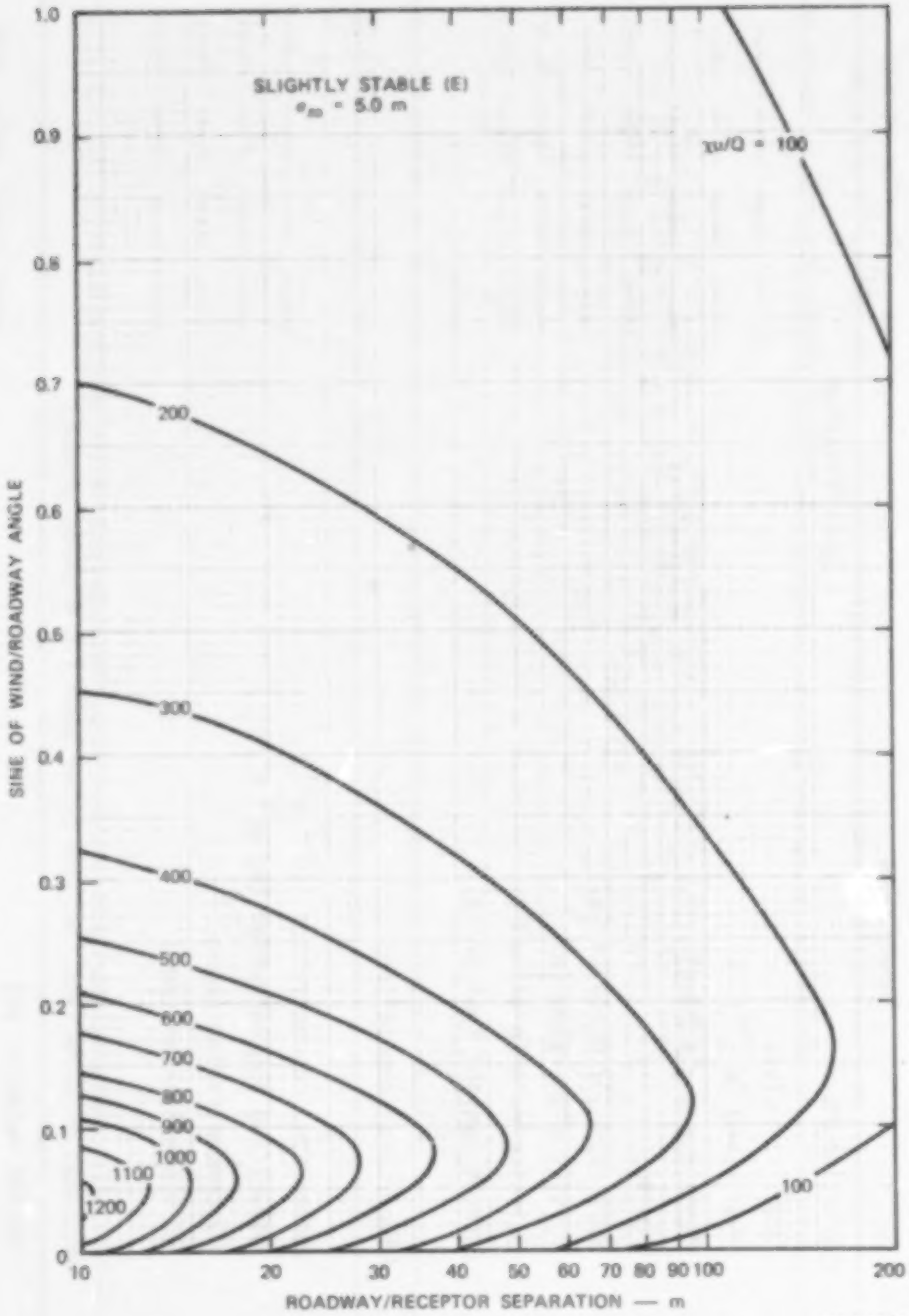

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FIGURE 9e VALUES OF $\chi u/Q$ ($10^{-3}m^{-1}$) FOR VARIOUS ROADWAY/RECEPTOR SEPARATIONS AND WIND/ROADWAY ANGLES; INFINITE LINE SOURCE

WORKSHEET 4--INFINITE LINE SOURCE CO DISPERSION ANALYSIS (see instructions following)

Project No.: ____________ Analyst____________

Site: ____________ Date: ____________

Step	Symbol	Input/Units	Traffic Stream			
		Basic inputs				
		Identification				
1	SC	Stability class				
2	U	Wind speed (m s^{-1})				
3	θ	Wind/road angle (deg)				
4		Sin θ				
5	x	Road receptor distance (m)				
6	σ_{z_o}	Initial dispersion (m)				
7	Q_o	Emission rate (g m^{-1} s^{-1})				
7a		Street canyon? (yes or no)				
		Dispersion computation				
8	$\chi U/Q$	Normalized concentration (10^{-3} m^{-1})				
9	χU	Normalized concentration (mg m^{-2} s^{-1})				
10	χ	CO concentration (mg m^{-3})				
11	χ	CO concentration (ppm)				
		Optional z-correction (heights other than 1.8 m above ground)				
12	z	Height receptor (m)				
13		z-correction factor				
14	χ'	CO concentration at height z (mg/m^3)				
15	χ'	CO concentration at height z (ppm)				

INSTRUCTIONS FOR COMPLETING WORKSHEET 4

Step Instructions

1 Determine stability class using Table 7.

2. Determine wind speed from procedures outlined in Figures 20-23.

3. Determine ambient wind direction from Figures 20-23; then compute the acute wind/roadway angle as illustrated in Figure 8.

4. Determine the sine of the wind/roadway angle.

5. Measure the road/receptor lateral separation (Figure 8).

6. Specify $\sigma_{z_0} = 1.5$ m for a smooth surface, or $\sigma_{z_0} = 5.0$ for a rough surface.

7. List the emission rate; see Worksheet 2 (line 18).

7a. Use Appendix C to determine if street canyon effects will dominate. If the test is positive Appendix C procedures supersede Steps 8-15. If the test is negative continue with Step 8.

8. Estimate the normalized concentration using Figure 9 and the following inputs; stability (line 1), sin θ (line 4), σ_{z_0} (line 6), and x (line 5).

9. Multiply line 8 by the emission rate (line 7) to obtain χu.

10. Divide line 9 by line 2 to obtain a computed value of the CO concentration (mg m^{-3}).

11. Multiply line 10 by .87 to obtain the value of the CO concentration in PPM.

Optional Correction for Receptor Height (not applicable to street canyon estimates)

12. Enter the height of the receptor (m).

13. Choose the proper figure (10 or 11) with stability class (line 1), and the initial dispersion, σ_{z_0} (line 6). With the road-receptor distance, x (line 5), and the height of the receptor: z (line 12) enter the figure and the z-correction factor. Enter this on line 13. (If $z > 18$, use $z = 18$ m).

14. Multiply line 13 and line 10. Enter on line 14.

15. Multiply line 13 and line 11. Enter on line 15.

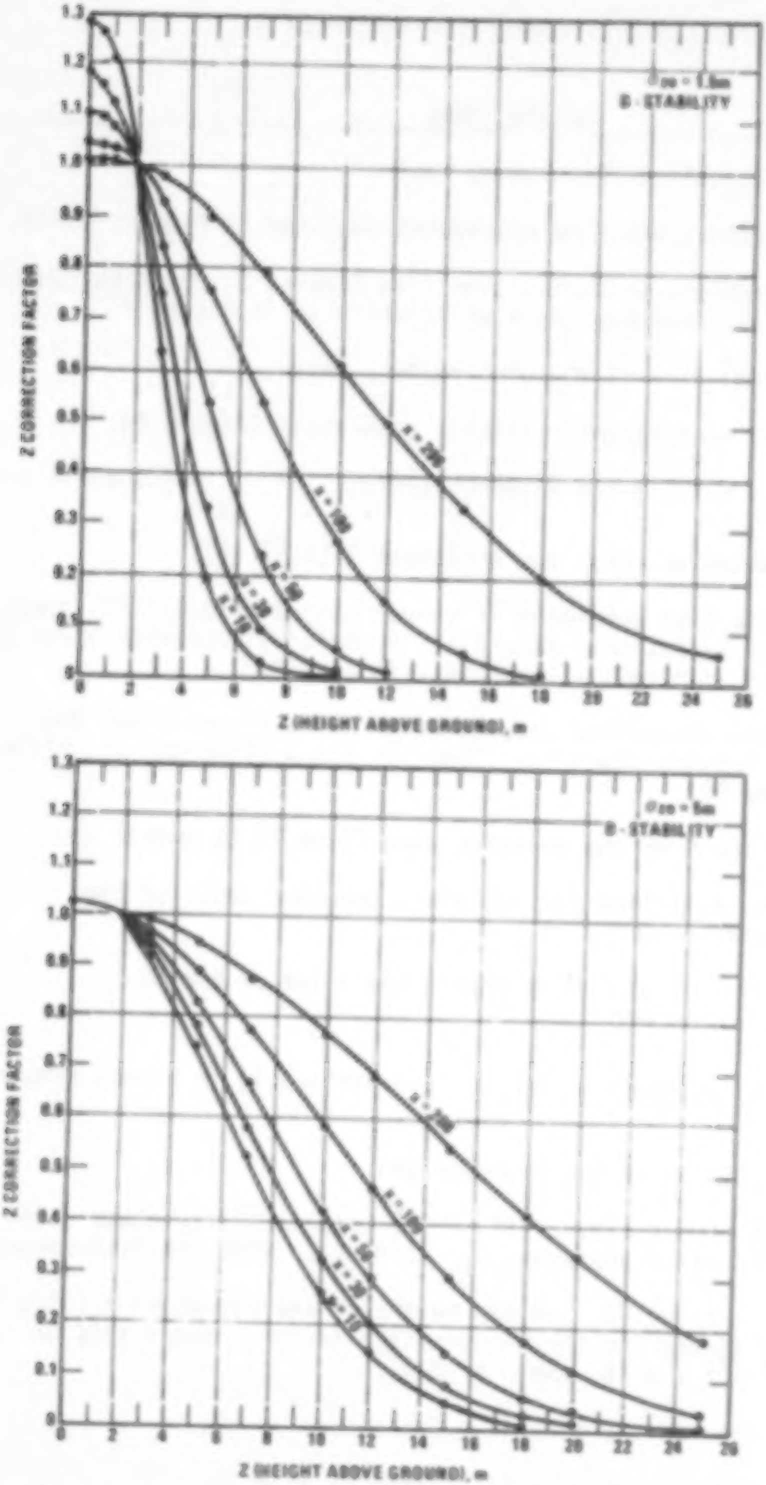


Figure 10. Correction factors for concentrations above ground level. D-stability.

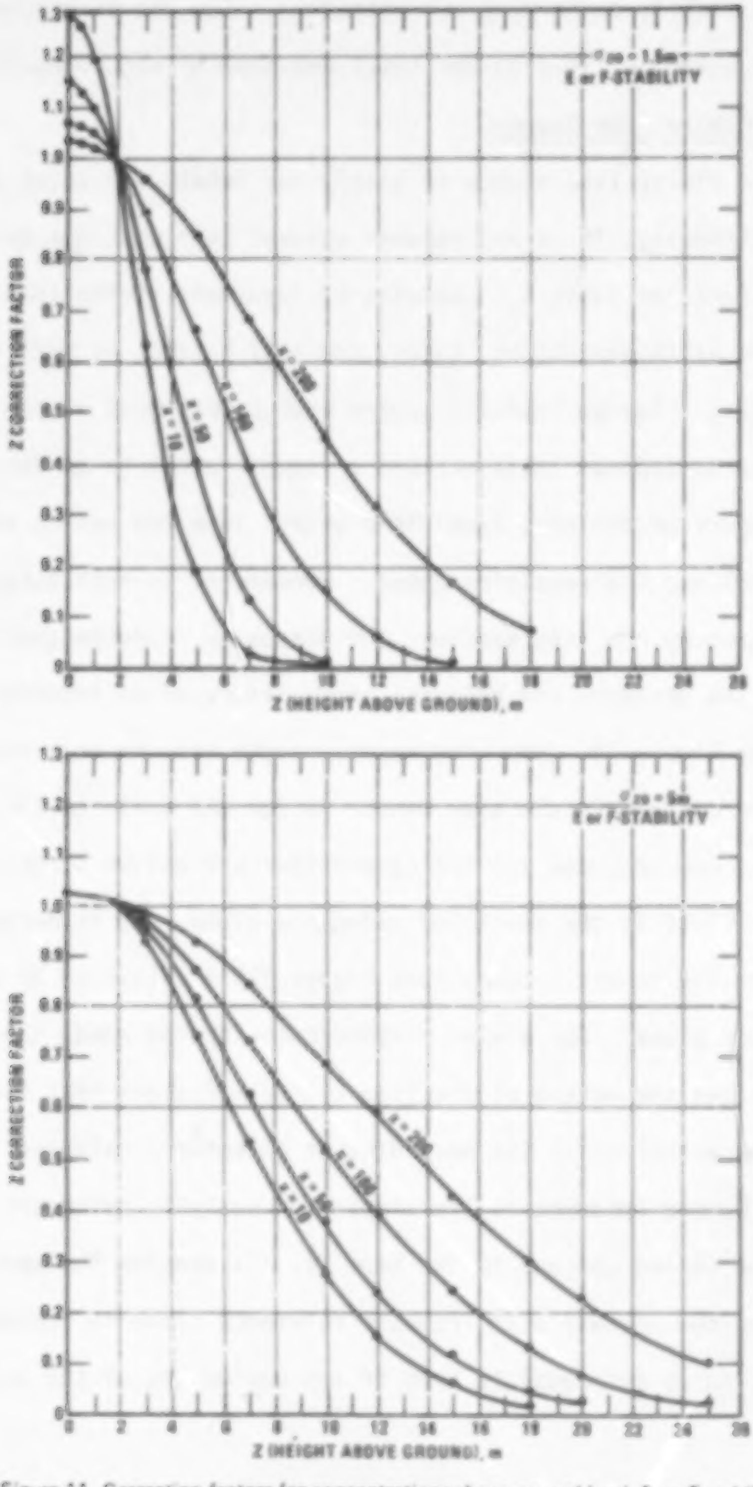


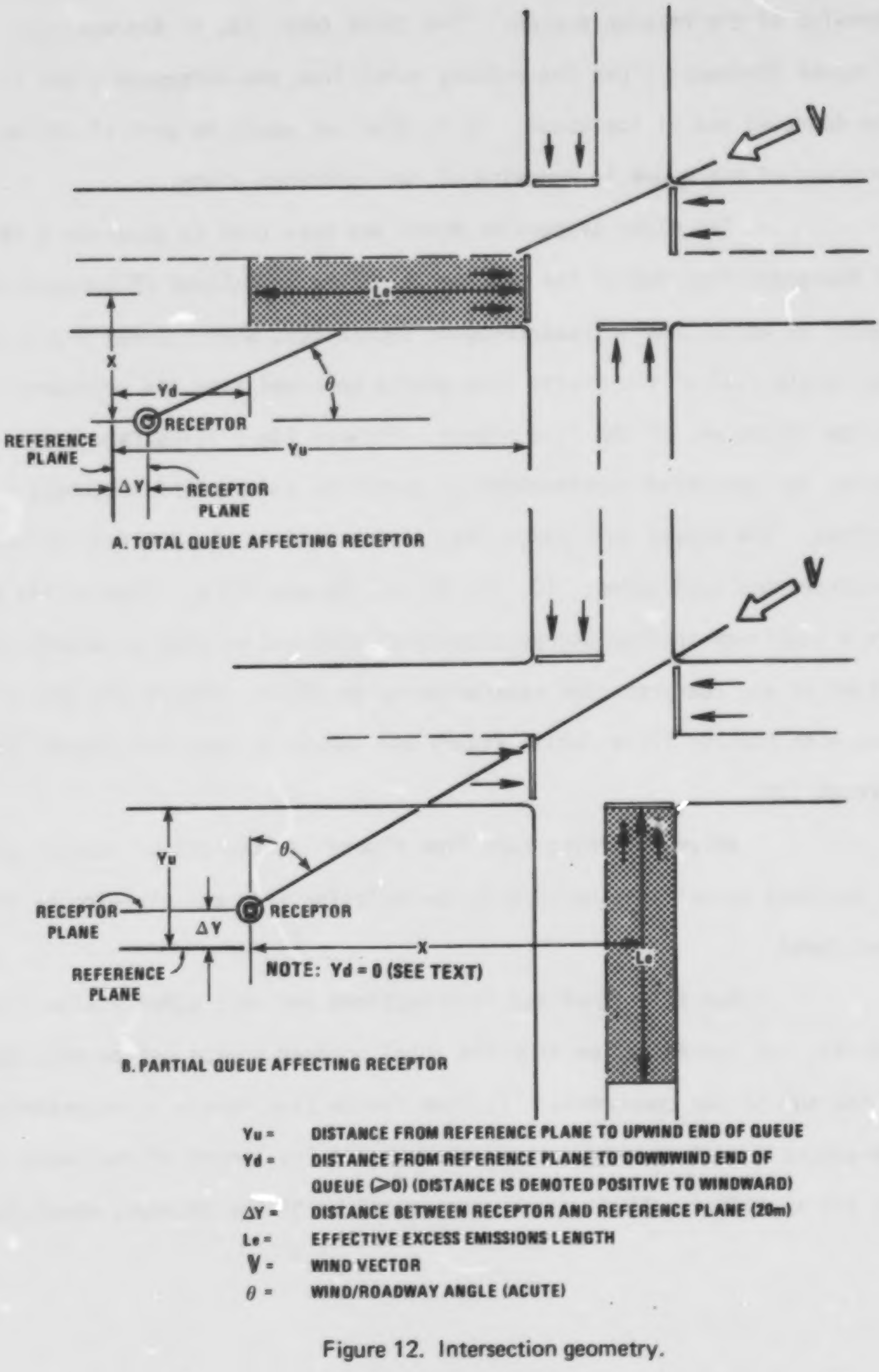
Figure 11. Correction factors for concentrations above ground level. E or F-stability.

area sources, and (3) background concentration. See the procedures outlined in Subsection III-E(3) for total one-hour or eight-hour CO impact.

b. Finite Line Source

A finite line source is simply any relatively short length of roadway--specifically, it is any roadway segment that does not satisfy the "minimum" criteria of Table 8. Examples of important finite line sources include queues at intersections, gates, and toll booths, as well as short roadway segments. For an indirect source evaluation, most finite line sources can be considered intersections or some analogy to an intersection. The finite aspect of the emissions field arises from the gating effect of traffic control and the resulting queue. Henceforth in this subsection, finite line sources and intersections are discussed interchangeably.

The geometry and relevant nomenclature of an intersection is illustrated in Figure 12. The wind/roadway angle and the road/receptor separation are defined in the same manner as for the infinite line source. However, three new concepts are introduced that are unique to the finite line source: First is the so-called reference plane. It is marked by a line perpendicular to the roadway (see Figure 12) and located 20 m downwind of the receptor plane. For a wind perpendicular to the road, the reference plane establishes the extent of the line source emissions that significantly affect the concentration at the most distant receptor considered (X = 200 m). Its use will become apparent as the dispersion analysis procedure is outlined. The second concept is the term Yu. It denotes the upwind distance along the roadway axis from the reference plane to upwind end of the queue; Yu is set equal to zero if the upwind end of the queue is


V
Le
X
Yd
θ
RECEPTOR
REFERENCE PLANE
Yu
ΔY
RECEPTOR PLANE
A. TOTAL QUEUE AFFECTING RECEPTOR
V
Yu
θ
RECEPTOR PLANE
RECEPTOR
ΔY
X
Le
REFERENCE PLANE
NOTE: Yd = 0 (SEE TEXT)
B. PARTIAL QUEUE AFFECTING RECEPTOR

Yu = DISTANCE FROM REFERENCE PLANE TO UPWIND END OF QUEUE

Yd = DISTANCE FROM REFERENCE PLANE TO DOWNWIND END OF QUEUE (≥0) (DISTANCE IS DENOTED POSITIVE TO WINDWARD)

ΔY = DISTANCE BETWEEN RECEPTOR AND REFERENCE PLANE (20m)

Le = EFFECTIVE EXCESS EMISSIONS LENGTH

V = WIND VECTOR

θ = WIND/ROADWAY ANGLE (ACUTE)

Figure 12. Intersection geometry.

downwind of the reference plane. The third term, Yd, is the upwind distance (measured along the roadway axis) from the reference plane to the downwind end of the queue. It is also set equal to zero if the downwind end of the queue is downwind of the reference plane.

The HIWAY dispersion model has been used to generate a series of nomograms that depict the dependence of the normalized CO concentration ($\chi U/Q$) on variations in road/receptor separation, wind/roadway angle, and the length (Yu) of the finite line source measured from the reference plane to the upwind end of the line source. Figures 13a-j illustrate these curves for specified combinations of stability and surface roughness conditions. The curves in Figures 13a-j correspond to six discrete values of road/receptor separation: 10, 15, 20, 40, 80 and 160 m. Figures 14a and b are a nonlinear interpolator/extrapolator that can be used to obtain $\chi U/Q$ values at any road/receptor separation up to 200 m. Figure 14a should be used with Figures 13a-b, while Figure 14b should be used for Figure 13c through 13j.

After obtaining $\chi U/Q$ from Figure 13, the actual concentration is obtained by multiplying $\chi U/Q$ by the emission rate and dividing by the wind speed.

For all signalized intersections and most other finite line sources, the concentration from the local roadway source can be considered as the sum of two components: (1) the finite line source as represented by the excess emissions rate emitted over the finite length of the queue (Le) and (2) an infinite line source representative of the through, nonstopping

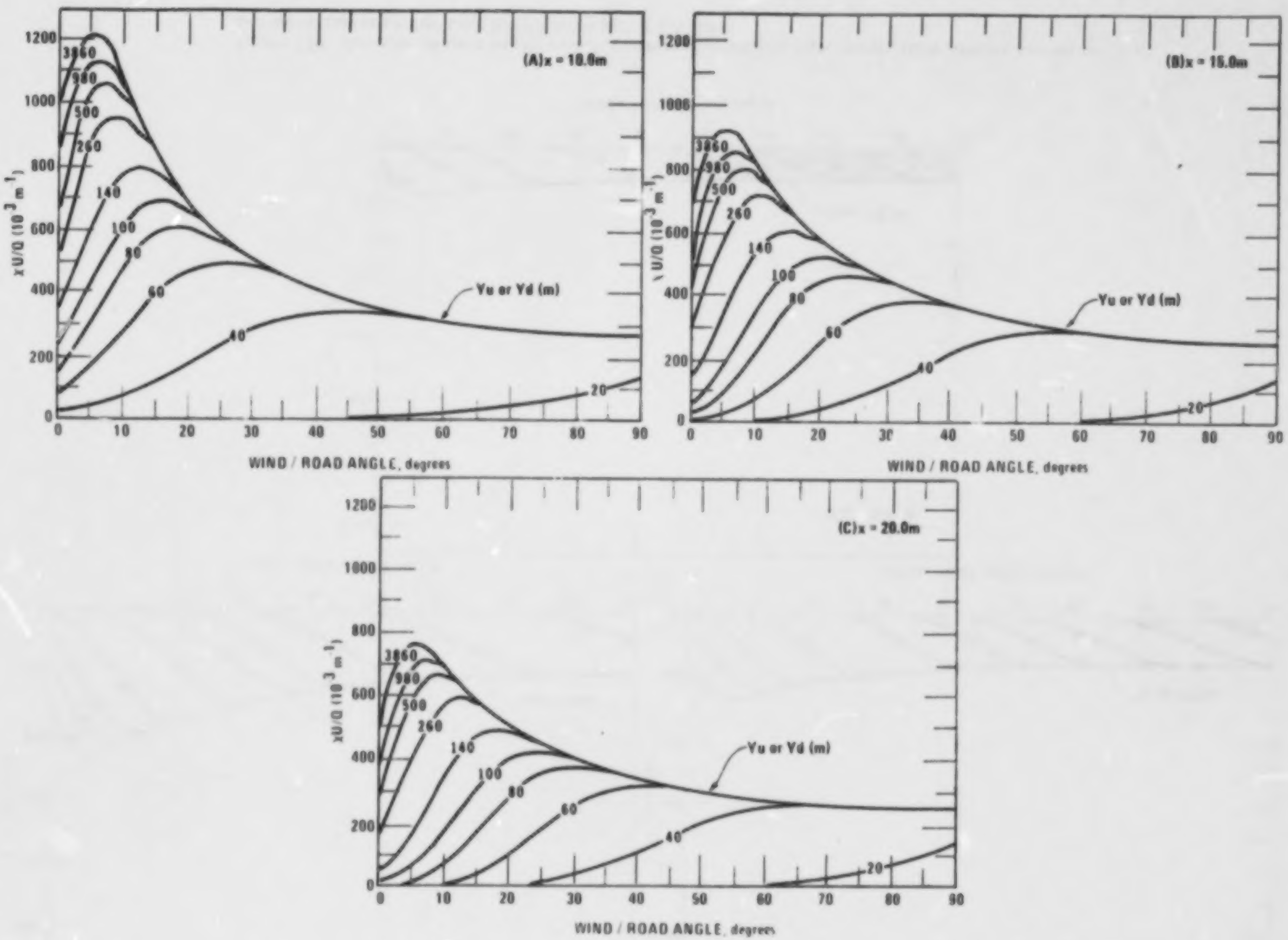


Figure 13a Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = D and σ_{zo} = 1.5 meters.

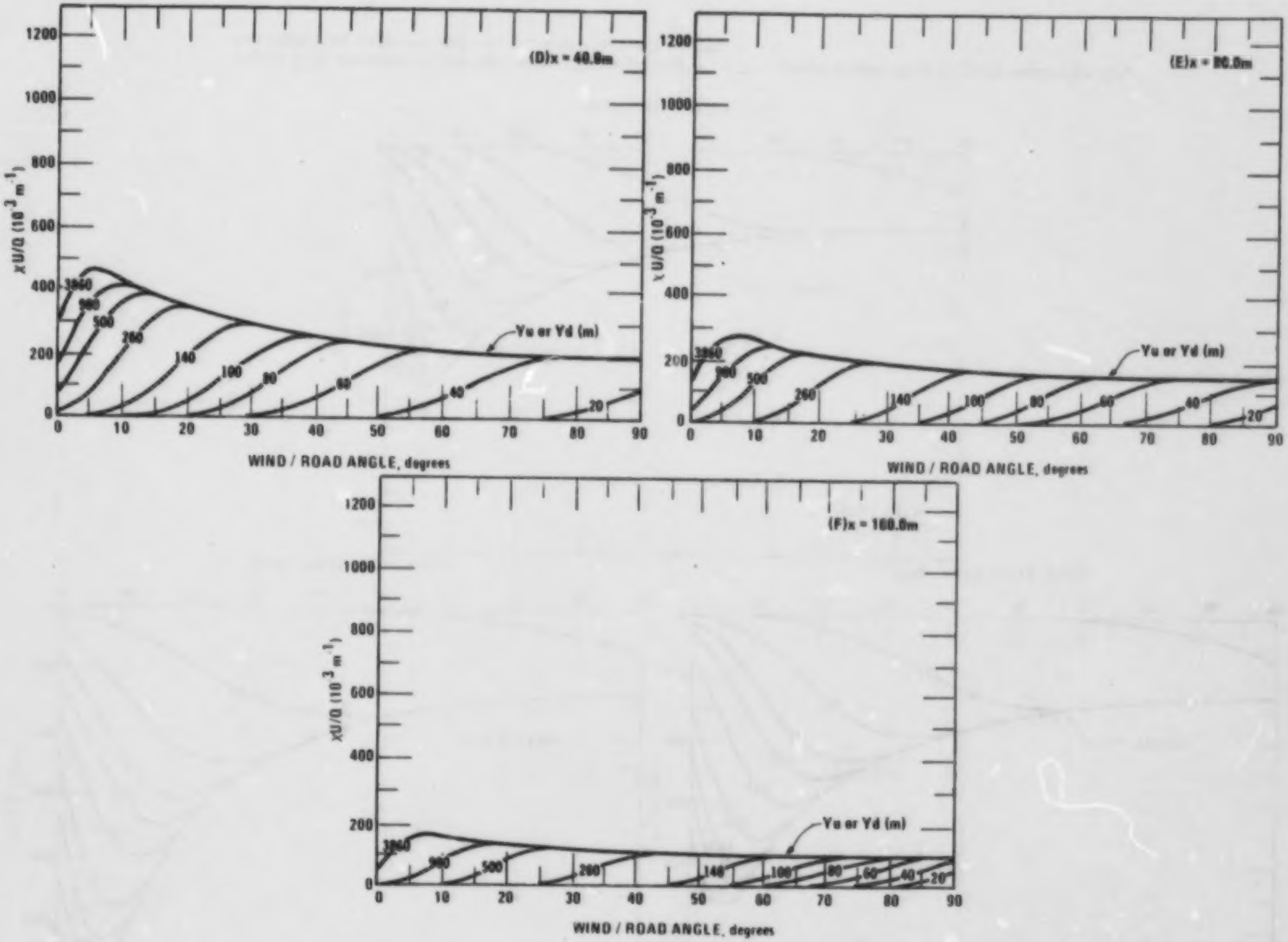


Figure 13b Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = D and σ_{zo} = 1.5 meters.

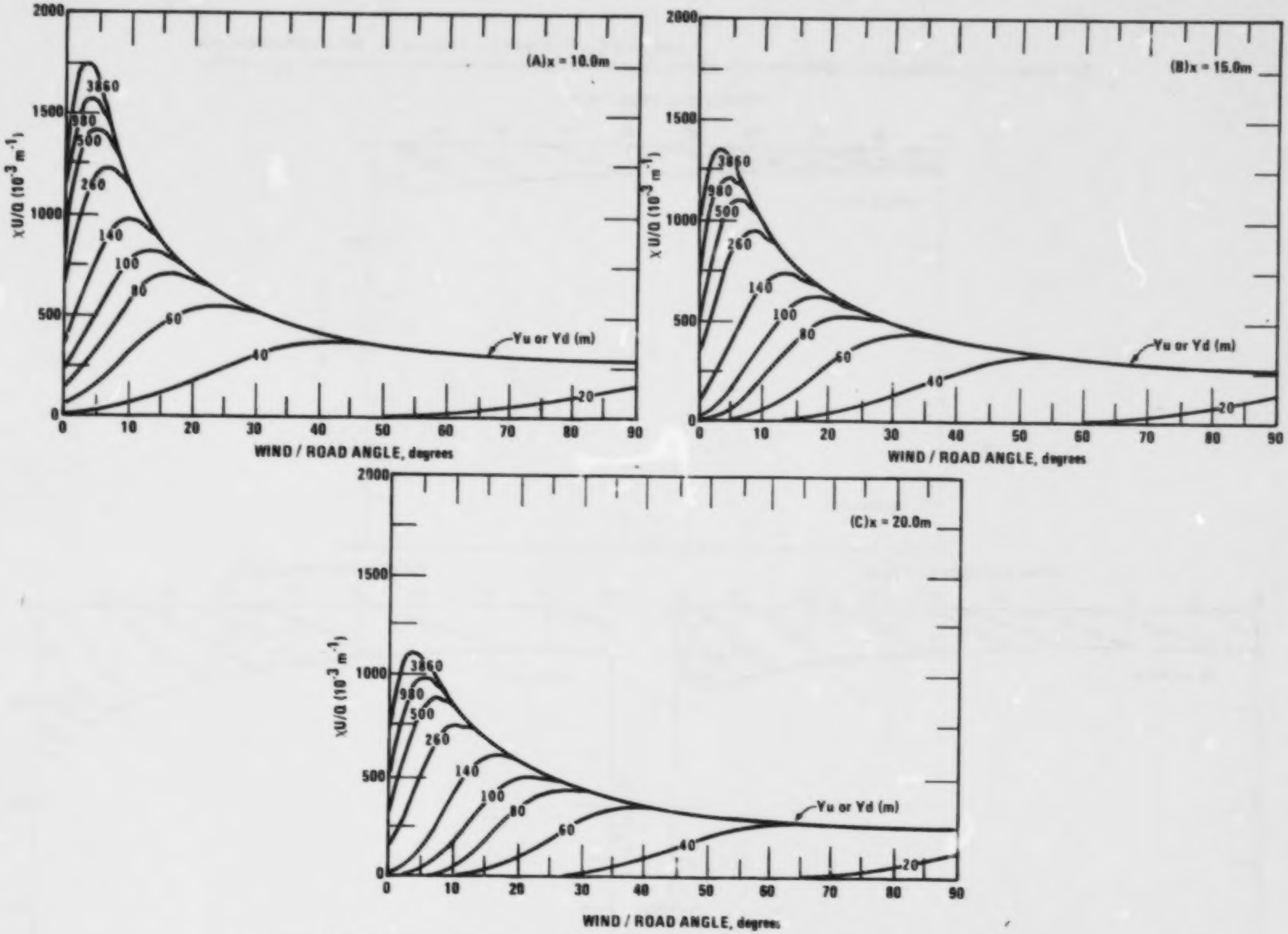


Figure 13c Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = E and σ_{zo} = 1.5 meters.

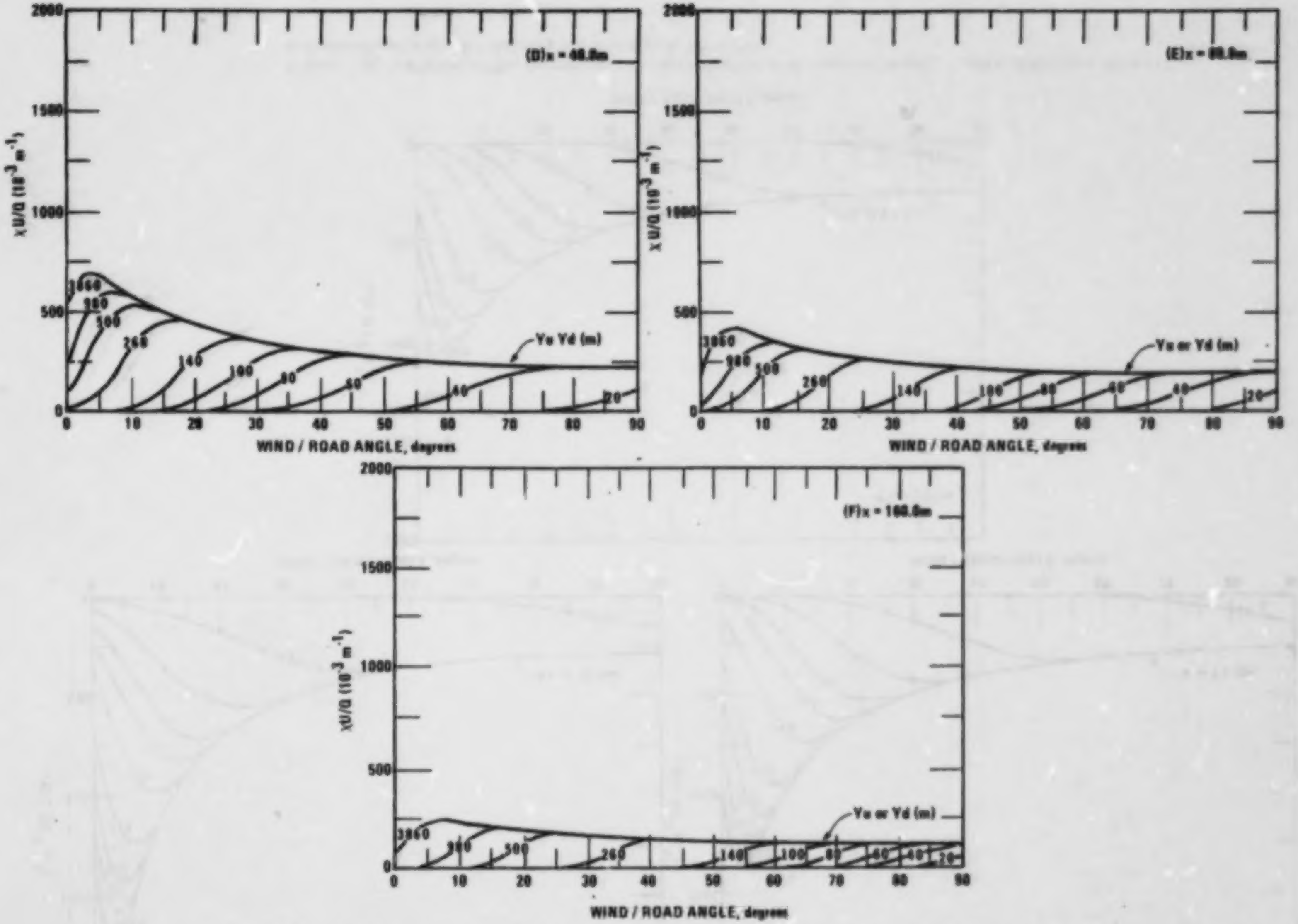


Figure 13d Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = E and σ_{zo} = 1.5 meters.

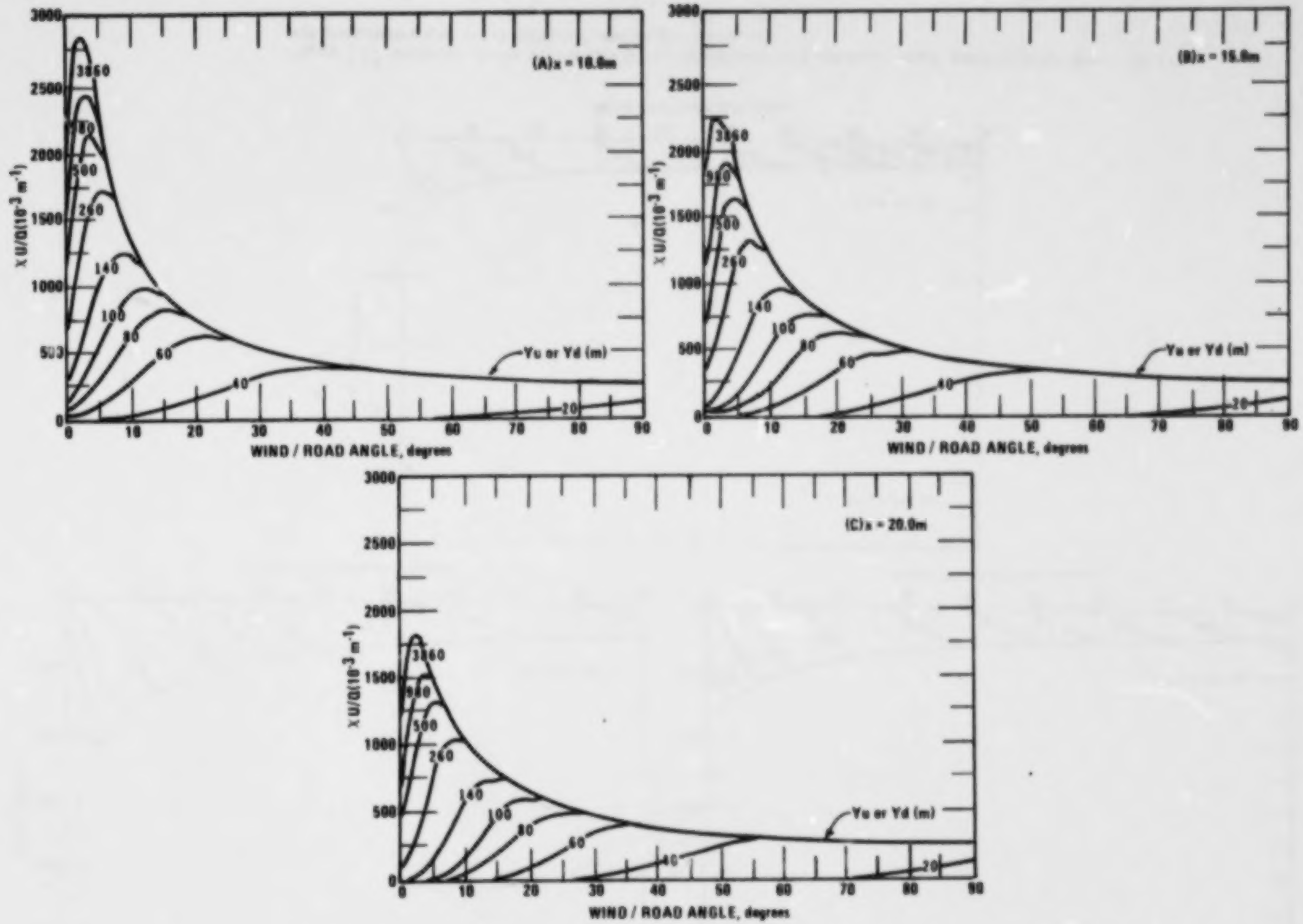


Figure 13e Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = F and σ_{zo} = 1.5 meters.

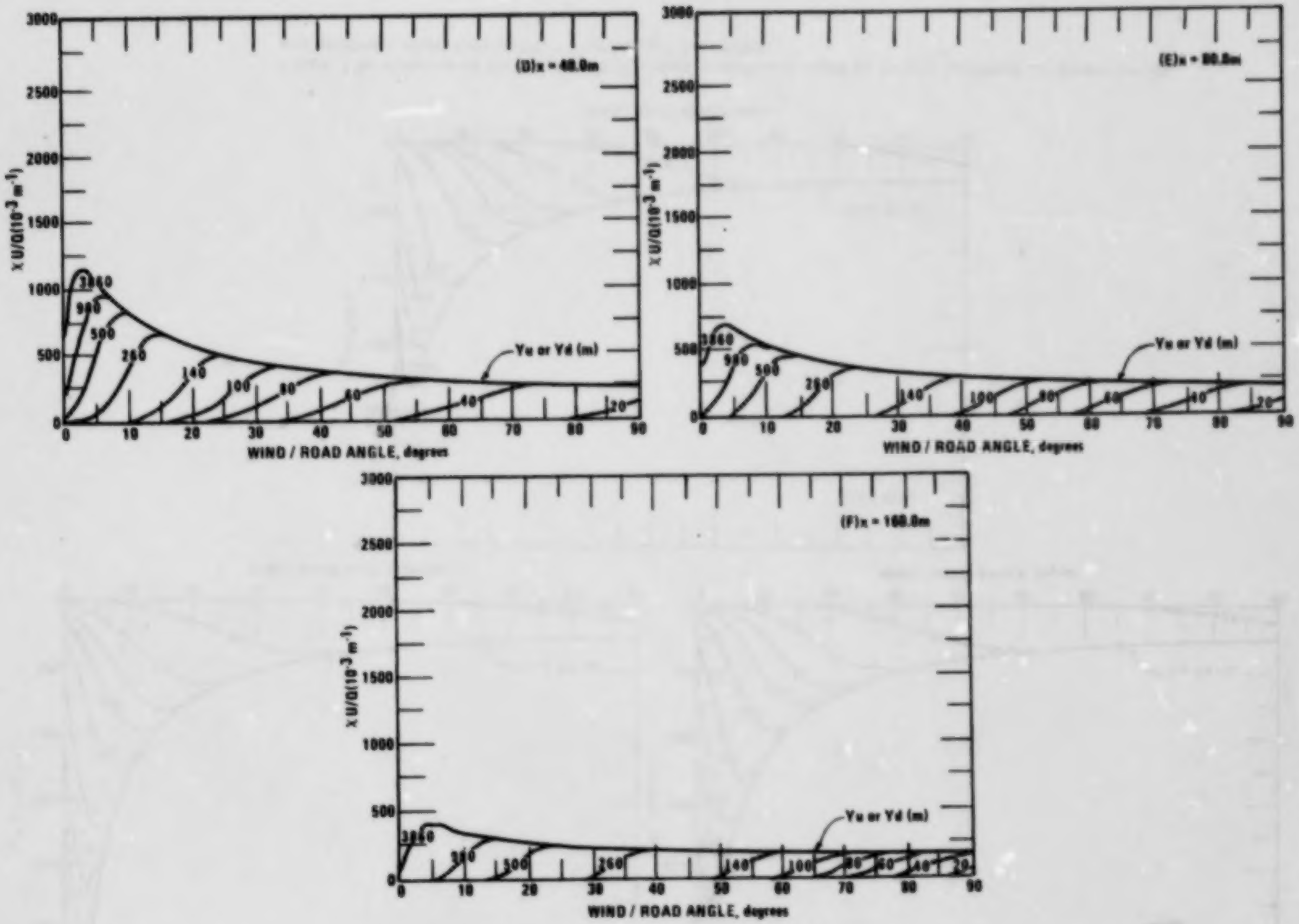


Figure 13f Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = F and σ_{zo} = 1.5 meters.

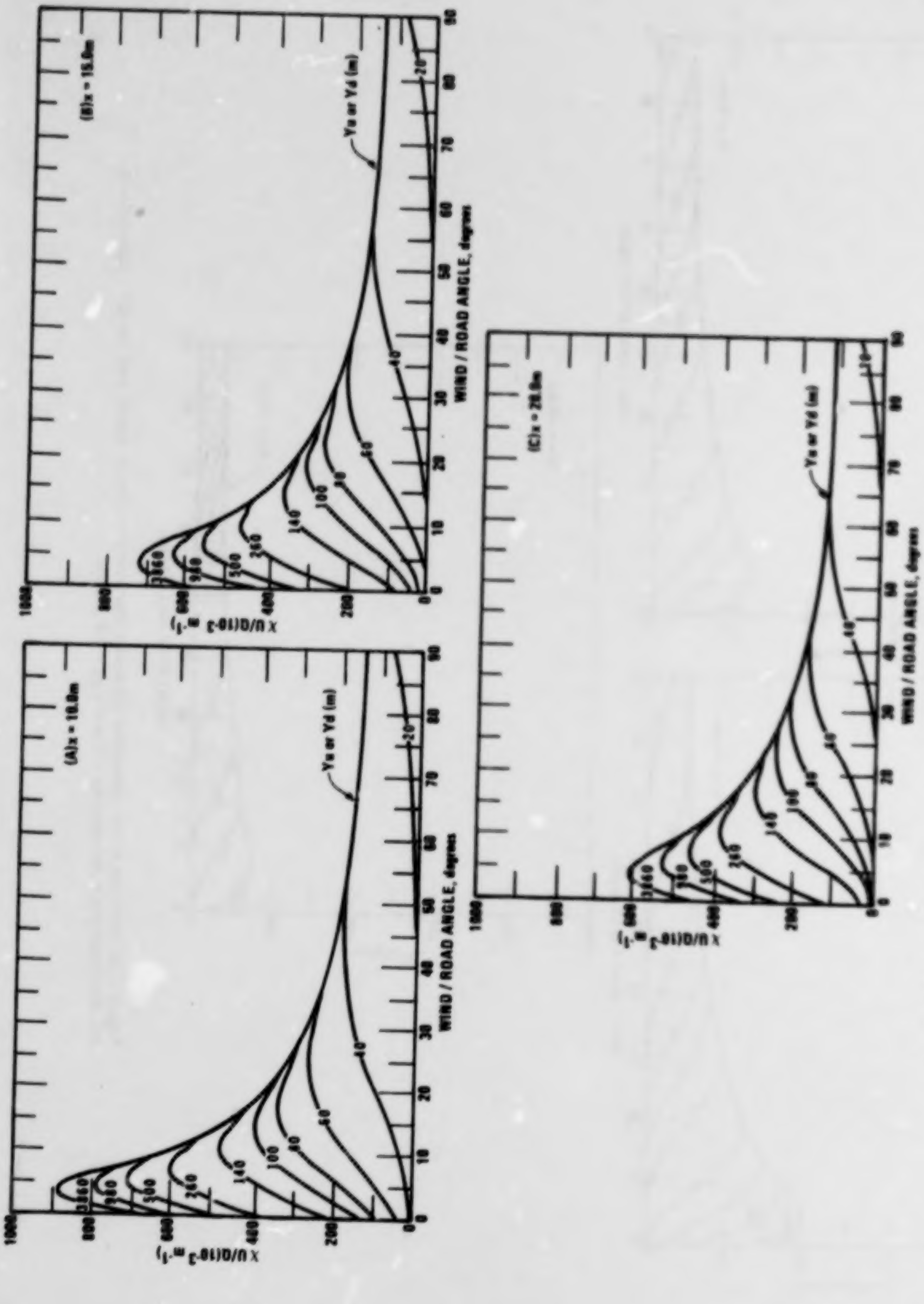


Figure 13g Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = D and σ_{zo} = 5.0 meters.

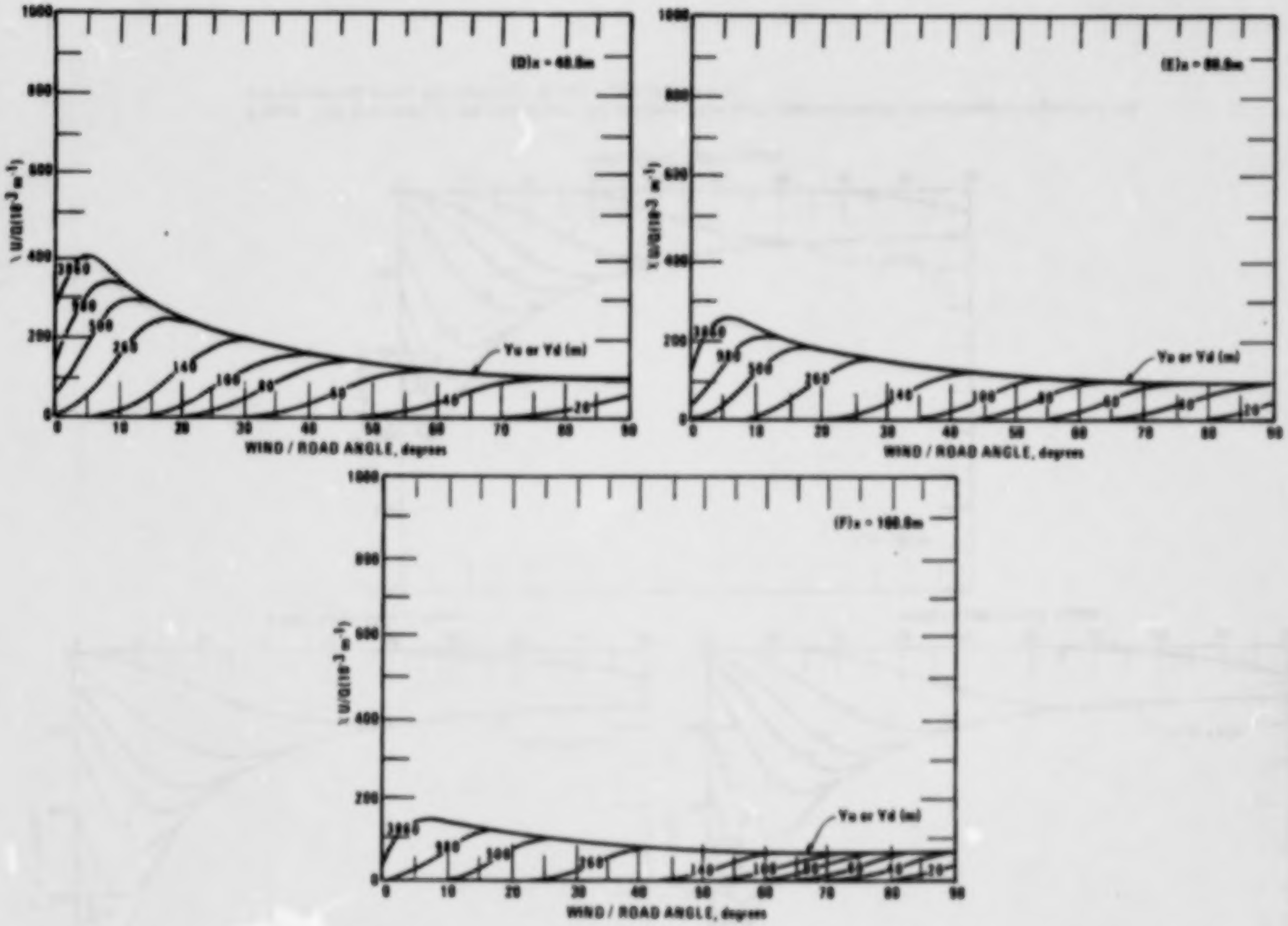


Figure [illegible] Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = D and σ_{zo} = 5.0 meters.

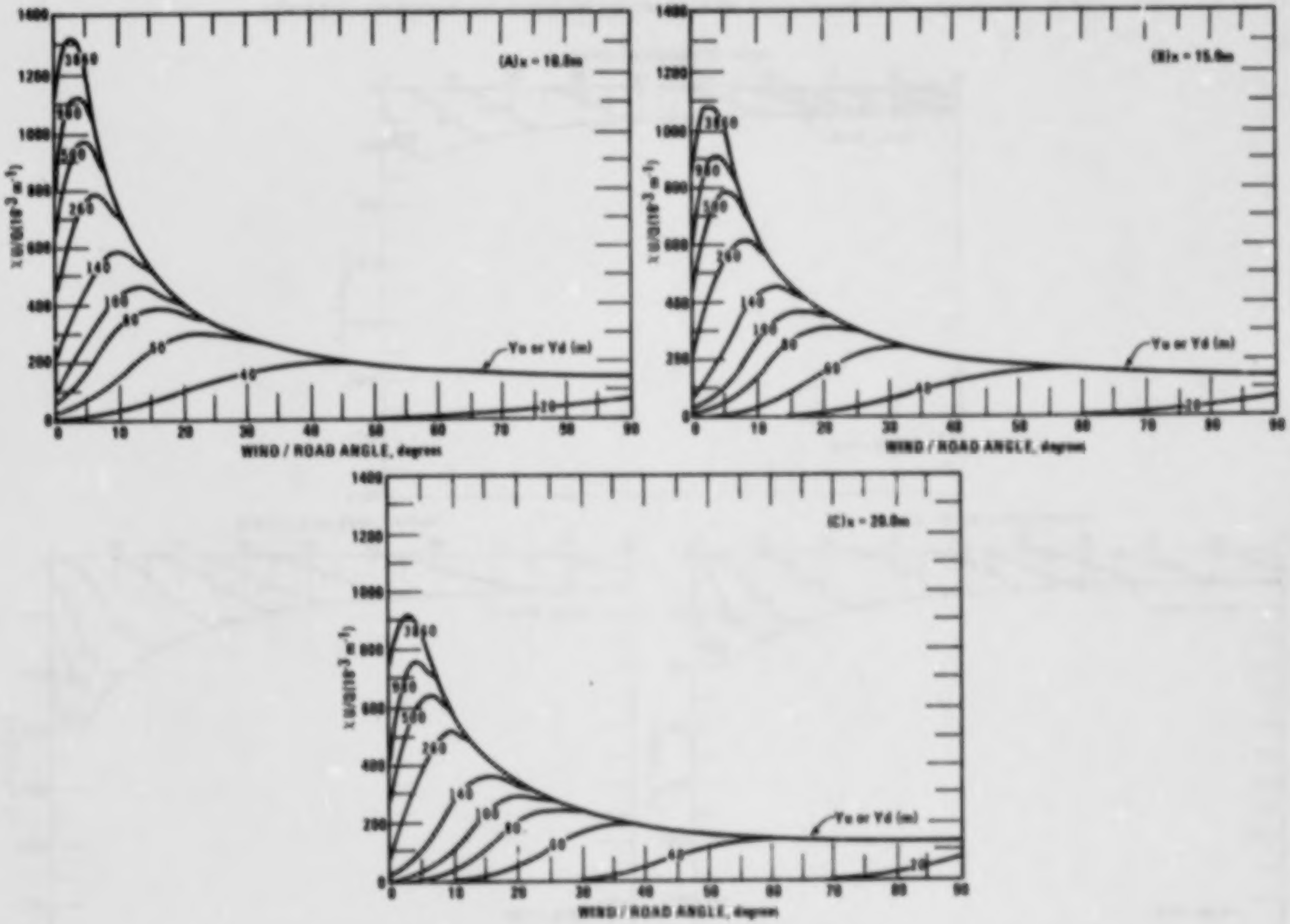


Figure 131 Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = E and σ_{zo} = 5.0 meters.

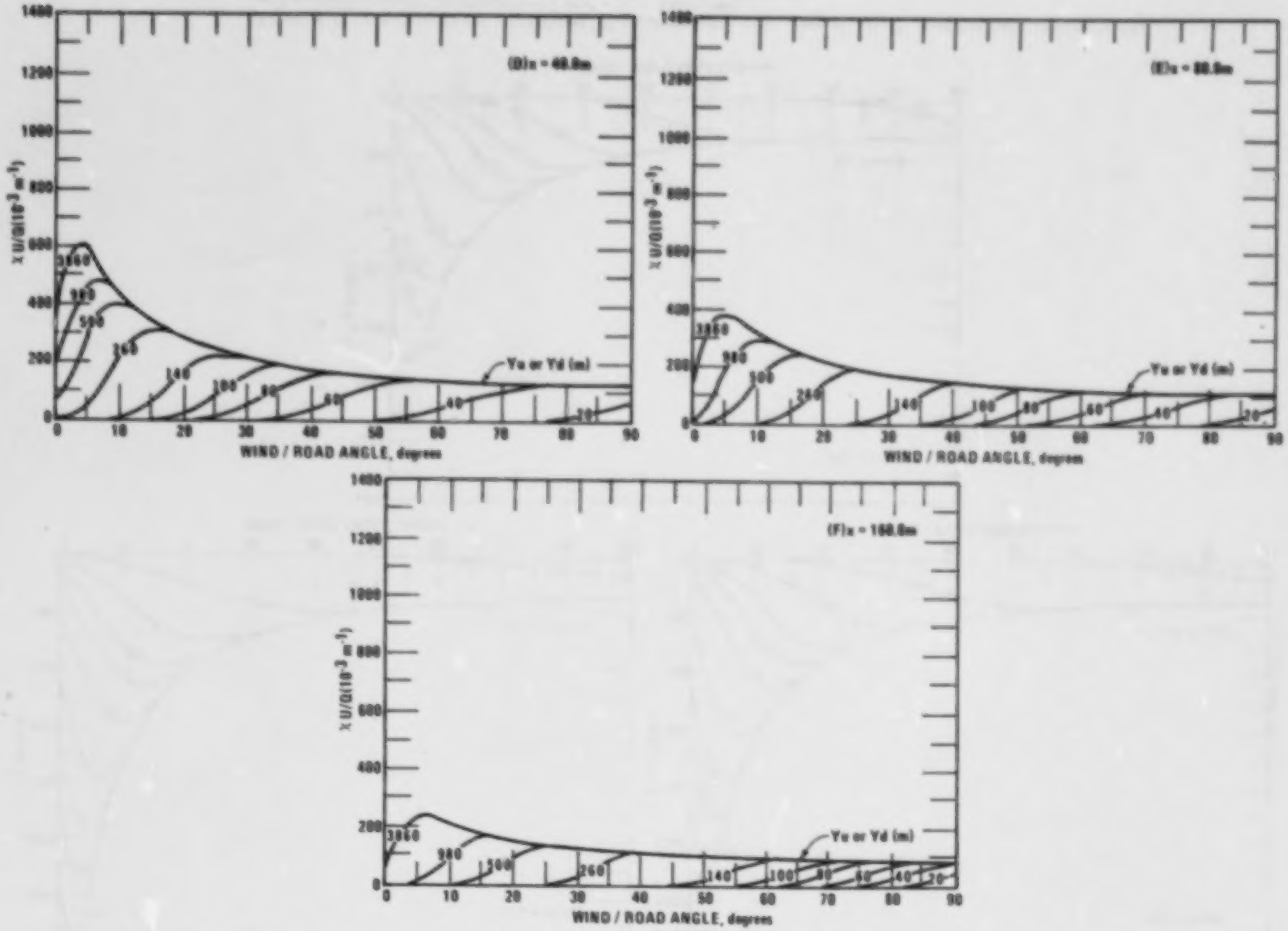
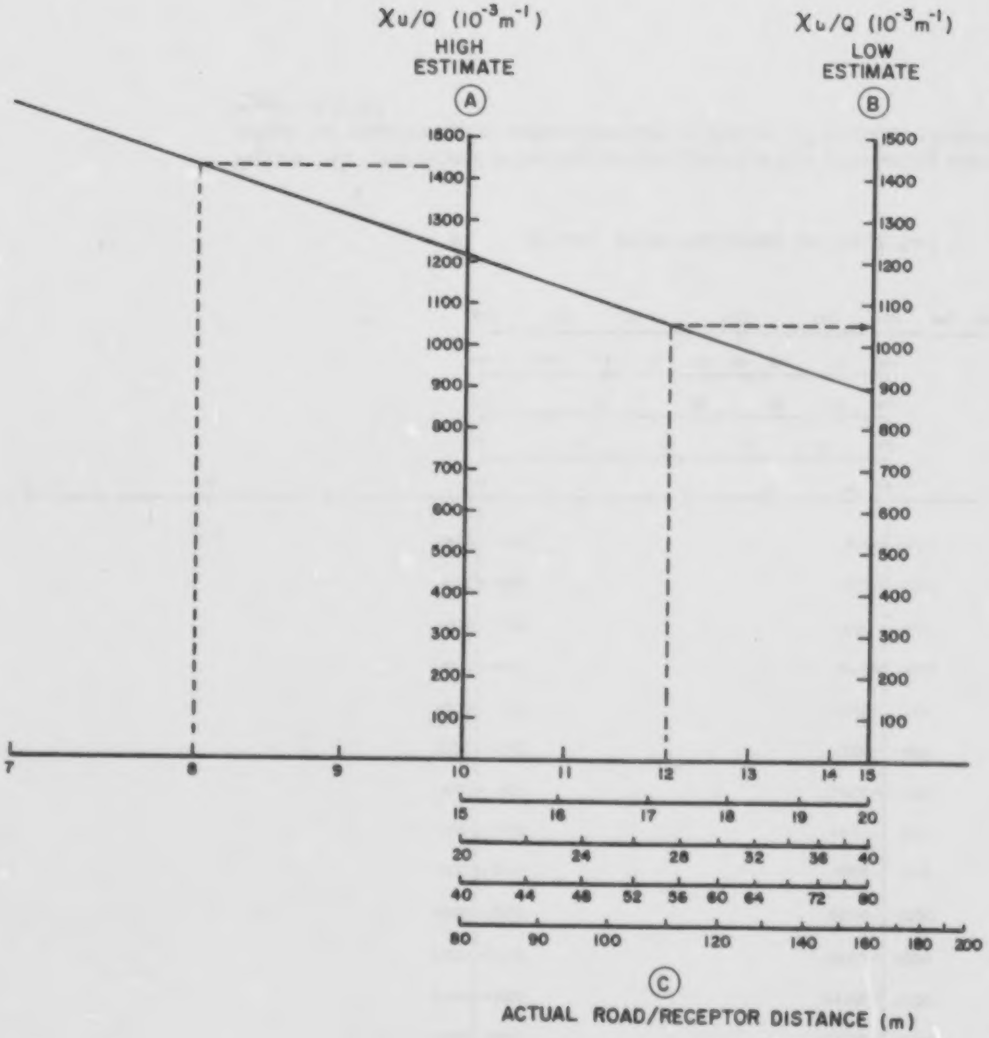


Figure 13j Variation of the normalized CO concentration with roadway length, road/receptor separation (x), and wind/road angle, for stability = E and σ_{zo} = 5.0 meters.



INSTRUCTIONS FOR USE:

This nomograph is used for calculating normalized concentration (xu/Q) estimates when the road/receptor distance is not one of the standard distances used in curves 12a.

- INTERPOLATION
  1. Select the (xu/Q) curves for road/receptor distances that bracket the actual value.
  2. Find the (xu/Q) values in each curve using the appropriate wind angle and excess emissions length.
  3. Enter the high estimate in column Ⓐ of the nomograph. Enter the low estimate in column Ⓑ. Connect the two estimates with a straight-edge.
  4. Find the actual road/receptor distance in rows Ⓒ.
  5. Move vertically to the straight-edge and horizontally to scale Ⓐ or Ⓑ. This is the interpolated value.
- EXTRAPOLATION
  1. If the actual road/receptor distance is less than 10 m or greater than 160 m, you must extrapolate.
  2. Calculate (xu/Q) from the curves of the TWO road/receptor distances closest to the actual
  3. Proceed to Steps 3, 4, and 5 above.

EXAMPLE:

GIVEN:

1. A concentration of 1220 at 10 m
2. A concentration of 900 at 15 m

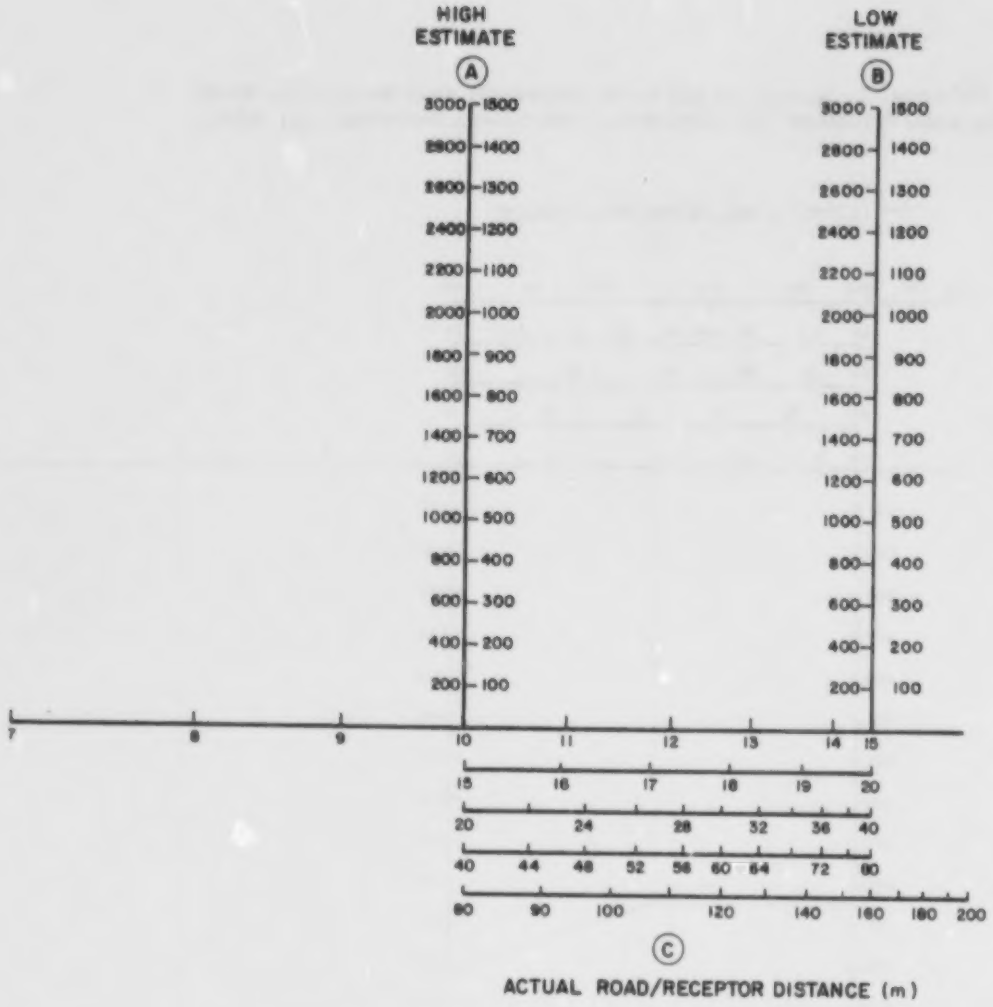
FIND:

1. The concentration at 12 m (interpolate)
2. The concentration at 8 m (extrapolate)

SOLUTION:

1. xu/Q at 12 m = 1050 × $10^{-3}m^{-1}$
2. xu/Q at 8 m = 1430 × $10^{-3}m^{-1}$

Figure 14a Nonlinear interpolator/extrapolator for calculating normalized concentration values for road/receptor separations up to 200 m. (D-stability and σ_{ZO} = 1.5 m).


HIGH ESTIMATE
Ⓐ
LOW ESTIMATE
Ⓑ
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
1500
1400
1300
1200
1100
1000
900
800
700
600
500
400
300
200
100
7
8
9
10
11
12
13
14
15
15
16
17
18
19
20
20
24
28
32
36
40
40
44
48
52
56
60
64
72
80
80
90
100
120
140
160
180
200
Ⓒ
ACTUAL ROAD/RECEPTOR DISTANCE (m)

INSTRUCTIONS FOR USE:

This nomograph is used for calculating normalized concentration ($\chi u/Q$) estimates when the road/receptor distance is not one of the standard distances used in curves 12b through 12c.

- INTERPOLATION
  1. Select the ($\chi u/Q$) curves for road/receptor distances that bracket the actual value.
  2. Find the ($\chi u/Q$) values in each curve using the appropriate wind angle and excess emissions length.
  3. Enter the high estimate in column Ⓐ of the nomograph. Enter the low estimate in column Ⓑ. Connect the two estimates with a straight-edge.
  4. Find the actual road/receptor distance in rows Ⓒ.
  5. Move vertically to the straight-edge and horizontally to scale Ⓐ or Ⓑ. This is the interpolated value.
- EXTRAPOLATION
  1. If the actual road/receptor distance is less than 10 m or greater than 180 m, you must extrapolate.
  2. Calculate ($\chi u/Q$) from the curves of the TWO road/receptor distances closest to the actual.
  3. Proceed to Steps 3, 4, and 5 above.

Figure 14b Nonlinear interpolator/extrapolator for calculating normalized concentration values for road/receptor separations up to 200 m. (For cases other than D-stability with $\sigma_{ZO} = 1.5$ m).

WORKSHEET 5 - INTERSECTION CO DISPERSION ANALYSIS

(see instructions following)

PROJECT NO.: ______________________ ANALYST: ______________________

SITE: ______________________ DATE: ______________________

LINE NO.	SYMBOL	INPUT/UNITS	TRAFFIC STREAM			
		BASIC INPUTS				
1	SC	STABILITY CLASS				
2	U	WIND SPEED ($m\ s^{-1}$)				
3	θ	WIND-ROAD ANGLE (deg)				
4	x	LATERAL DISTANCE (m)				
5	Yu	MAXIMUM LONGITUDINAL DISTANCE (m)				
6	Yd	MINIMUM LONGITUDINAL DISTANCE (m)				
7	σ_{z_0}	INITIAL DISPERSION (m)				
8	Qe	EXCESS EMISSIONS RATE ($g\ m^{-1}\ s^{-1}$)				
9	Qf	FREE FLOW EMISSIONS RATE ($g\ m^{-1}\ s^{-1}$)				
9a		STREET CANYON? YES OR NO				
		DISPERSION ANALYSIS				
10	χUQ^{-1}	NORMALIZED CONCENTRATION ($10^{-3}\ m^{-1}$) FREE FLOW				
	Qf	ENTER LINE 9	×	×	×	×
11	χU	NORMALIZED CONCENTRATION ($mg\ m^{-2}\ s^{-1}$)				
	U	ENTER LINE 2	÷	÷	÷	÷
12	χ	CO CONCENTRATION ($mg\ m^{-3}$) THROUGH EMISSIONS				
13	χUQ^{-1}	NORMALIZED CONCENTRATION (FOR Yu)				
	Qe	ENTER LINE 8				
14	χU	NORMALIZED CONCENTRATION ($mg\ m^{-2}\ s^{-1}$)				
	U	ENTER LINE 2	÷	÷	÷	÷
15	χ	CO CONCENTRATION--"MAXIMUM QUEUE"				
16	χUQ^{-1}	NORMALIZED CONCENTRATION (FOR Yd)				
	Qe	ENTER LINE 8	×	×	×	×
17	χU	NORMALIZED CONCENTRATION ($mg\ m^{-1}\ s^{-1}$)				
	U	ENTER LINE 2	÷	÷	÷	÷
18	χ	CO CONCENTRATION--"IMAGINARY QUEUE"	+	+	+	+
19	χ	CO ($mg\ m^{-3}$) TOTAL				
20	χ	CO CONCENTRATION (ppm)--TOTAL				
		OPTIONAL z-CORRECTION (HEIGHTS OTHER THAN 1.8 m ABOVE THE GROUND)				
21	z	HEIGHT OF RECEPTOR (m)				
22		z CORRECTION FACTOR				
23	χ'	CO CONCENTRATION AT HEIGHT z (mg/m^{-3})				
24	χ'	CO CONCENTRATION AT HEIGHT z (ppm)				

INSTRUCTIONS FOR COMPLETING WORKSHEET 5

Step	Instructions
1	Determine stability class using Table 7.
2	Determine wind speed from procedures outlined in Figures 20-23.
3	Determine ambient wind direction from Figures 20-23; then compute the acute wind/roadway angle as illustrated in Figure 12.
4	Measure the road/receptor lateral separation (Figure 12).
5	Determine the distance Yu (measured parallel to the roadway) upwind from the reference plane to the upwind end of queue (Figure 12, note: add 20m to Yu as shown).
6	Determine the distance Yd (measured parallel to the roadway) upwind from the reference plane to the downwind end of queue (Figure 12, note: add 20 m to Yd as shown); Yd $\geq$ 0.
7	Specify σ_{z_o} = 1.5 m for a smooth surface, and σ_{z_o} = 5.0 m for a rough surface.
8	List the excess emissions rate; see Worksheet 2 (line 17).
9	List the free-flow emissions rate; see Worksheet 2 (line 18).
9a	Use Appendix C to see if street canyon effects will dominate at the receptor. If the test is positive Appendix C calculations should be carried out. The estimate will later be compared to the total estimate in line 19 or 23 (mg/m^3). If no street canyon continue with line 10.
10	Estimate normalized concentration for infinite line source from Figure 9 using line 4 and the sine of line 3.
11	Multiply line 10 by line 9 and enter.
12	Divide line 11 by line 2 which gives the computed value of the CO concentration (mg^{-3}) for free flow emissions.
13	Estimate the normalized concentration for a fine line source using line 5 (Yu), line 3 (θ), line 4 (x), line 1 (SC), and line 7 (σ_{z_o}) to enter Figures 13 and 14.
14	Multiply line 13 by line 8 and enter.
15	Divide line 14 by line 2 (wind speed, U), which gives the computed value of the CO concentration (mg m^{-3}) for a "maximum" queue extending over Yu.

16 Estimate the normalized concentration for a finite line source using line 6 (Yd), line 3 (θ), line 4 (x) line 1 (SC), and line 7 (σ_{z_0}) to enter Figures 13 and 14.

17 Multiply line 16 by line 8 and enter.

18 Divide line 17 by line 2 (wind speed, U) which gives the computed value of the CO concentration (mg m^{-3}) for an "imaginary" queue extending over Yd. This is entered as a negative contribution.

19 Add lines 12, 15 and 18 to obtain the total CO concentration (mg m^{-3}). Add estimates for all approaches (compare to line C-7 in Appendix C if line 9a showed a street canyon effect possible and $z = 1.8$ m. Enter the higher of line C-7 or the total of line 19. If $z \neq 1.8$ m continue with lines 21-24).

20 Multiply by the conversion factor (0.87) to obtain the CO concentration in ppm.

Optional Correction for Receptor Height (Not Applicable to Street Canyon Estimates)

---

21 Enter the height of the receptor (m).

22 Choose the proper figure (10 or 11) with stability class (line 1) and the initial dispersion, σ_{z_0} (line 7). With the road-receptor distance, x (line 4), and the height of the receptor, z (line 21) enter the figure and find the z-correction factor. Enter this on line 22. If $z > 18$, let $z = 18$ m).

23 Multiply line 22 by line 19 for each approach. Add estimates for all approaches (compare to line C-7 in Appendix C if line 9a showed street canyon effect possible. Enter the higher of line C-7 or the total of line 23).

24 Multiply line 23 by 0.87.

vehicles. Accordingly, a dual analysis is required--one for each component. Worksheet 5 summarizes the step-by-step procedure for computing hourly concentrations. To obtain the total estimated CO concentration at the receptor, CO contributions by other local line and area sources need to be added along with the background contribution. The procedure for estimating the eight-hourly impact is outlined in Section III-E(3).

c. Area Source

Distributed emissions at or near the indirect source can be agglomerated to simulate an extended area source of uniform emissions (Qa, g m^{-2} s^{-1}). An area source dispersion formulation based on the integration of the Gaussian plume equation has been used by Ludwig et al. (1970) in the APRAC simulation model. That approach is used here. The method requires that the area source occupy a pie-shaped area focused on the receptor. For best estimates the area source should align with and extend to 23° on both sides of the wind vector that passes through the receptor. Example 3 in Chapter IV-3 shows a way of meeting this criteria.

The Gaussian plume equation for an area source simplifies to

$$\frac{\chi u}{Qa} = \frac{0.8}{a\,(1-b)}\; r_u^{1-b} - r_d^{1-b} \tag{35}$$

where r_u is the effective distance from the receptor to the upwind edge of the area source, and r_d is the effective distance to the downwind edge. More specifically, r is defined as

$$r \equiv x + x_0, \tag{36}$$

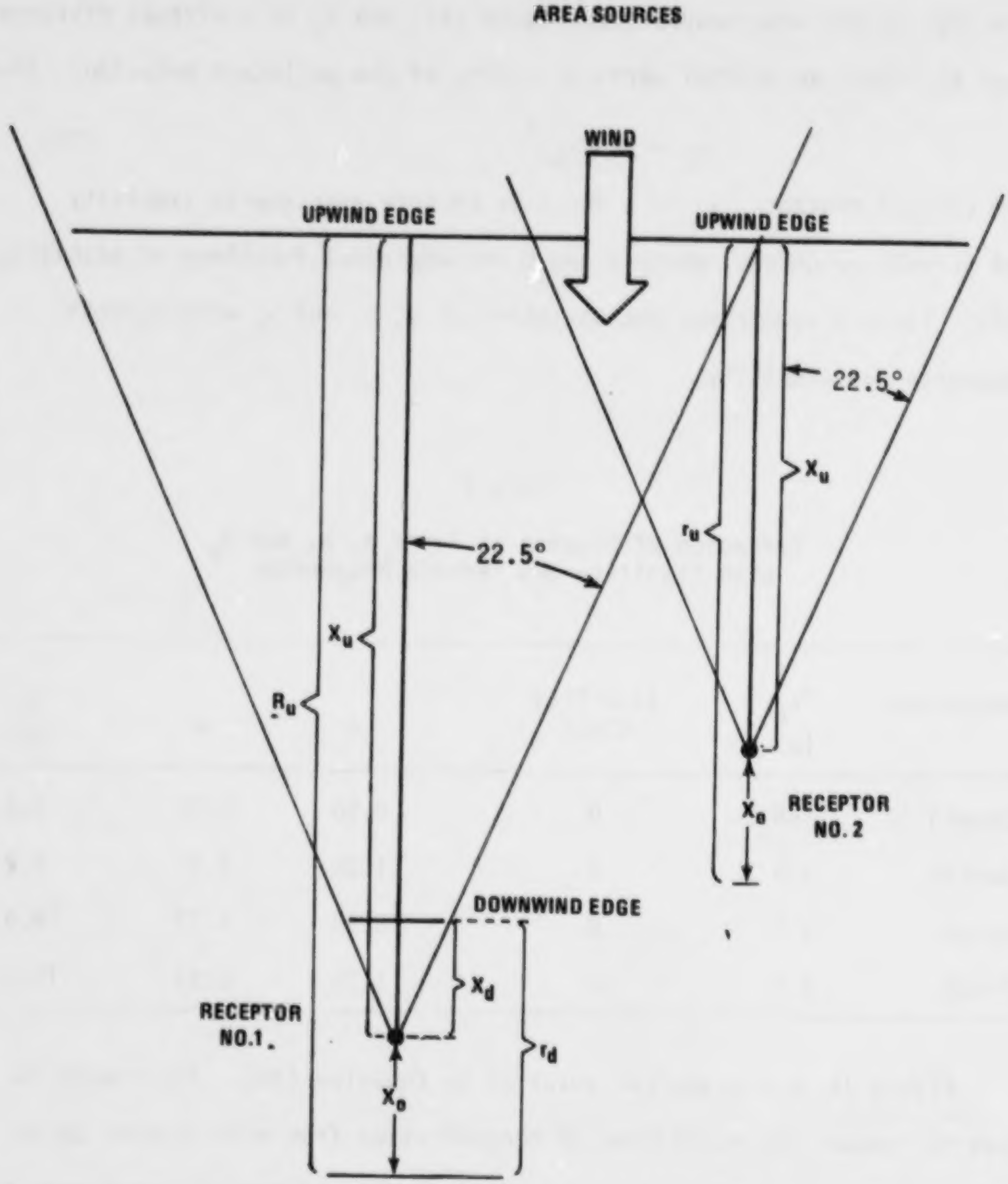

AREA SOURCES
WIND
UPWIND EDGE
UPWIND EDGE
22.5°
22.5°
X_u
r_u
X_o
RECEPTOR NO. 2
X_u
R_u
DOWNWIND EDGE
X_d
RECEPTOR NO.1
r_d
X_o

Figure 15 Graphical representation of the relationship between the actual distance (x) and the effective distance (r) for an area source.

where x is the actual distance along the wind vector from the receptor to the edge of the area source (see Figure 15), and x_o is a virtual distance used to affect an initial vertical mixing of the pollutant emission. Thus,

$$\sigma_z = a(x+x_o)^b \ . \tag{37}$$

The virtual distance, x_o is a function of both atmospheric stability and terrain roughness, while a and b are empirical functions of stability only. Table 9 summarizes the variation of a, b, and x_o with terrain roughness and stability.

Table 9

Variation of Dispersion Terms a, b, and x_o With Stability and Terrain Roughness

Roughness	σ_{z_o} (m)	Stability Class	a	b	x_o (m)
Smooth	1.5	D	0.50	0.77	4.2
Smooth	1.5	E	1.35	0.51	1.2
Rough	5.0	D	0.50	0.77	19.9
Rough	5.0	E	1.35	0.51	13.0

Figure 16 is a graphical solution to Equation (35). The figure is used to compute the normalized CO concentration from area sources up to 500 m distant. Beyond that range, the influence of typical worst-case mixing lids may become important (a separate treatment for those cases is given at the end of this section). Normalized concentrations are computed in the following sequence:

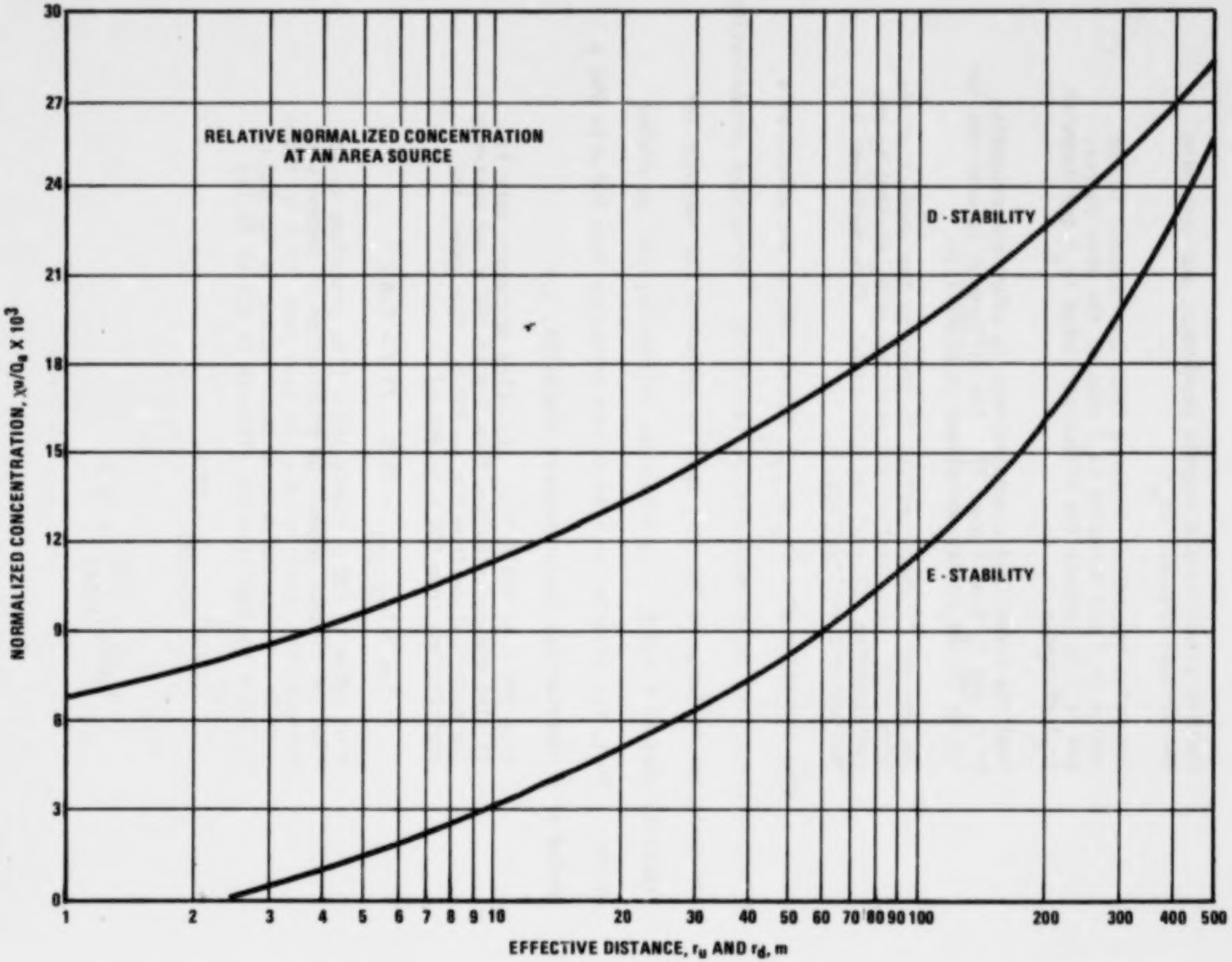


Figure 16. Variation of normalized concentration with stability and effective distance of the area source.

- Specify stability and terrain roughness, and determine the virtual distance (x_o).

- Determine the actual distance from the receptor to the upwind (x_u) and downwind (x_d) edge of the area source; add (x_o) to compute the effective upwind (r_u) and downwind (r_d) distances.

- Read the normalized concentrations (χ u/Qa) corresponding to r_u and r_d from Figure 17. The difference between the two values is the actual normalized concentration.

- The ambient CO concentration is given by the product of the normalized concentration and emission rate, divided by the corresponding value of the wind speed. (See Worksheet 6 for complete instructions.

When the value of r (r_u or r_d) exceeds 500 m, the effect of a limiting mixing lid (H, m) must also be considered. Normalized concentrations are estimated in Table 10 for four sectors upwind of the receptor for stability classes D and E. The estimates include typical low mixing heights. Thus, for example, an area source extending from 200 m to 900 m upwind of a receptor has (under neutral stability, D,):

from 200 m to 500 m (Fig. 16)--[The procedure here is to find the contribution from 0 m to 500 m and subtract the contribution from 0 m to 200 m. This gives the contribution from 200 m to 500 m]

$$(\chi U/Qa)_{200-500} = 29.0 - 23.2 = 5.8$$

from 500 m to 900 m (Table 10) - [The procedure is to find the closest segment or combination of segments to estimate the contribution. In this case 500 m to 1000m = 15.74. Since the desired contribution is for 500 m to 900 m the contribution estimate is 80% of 15.74]

$$(\chi U/Qa)_{500-900} = 12.6$$

Thus,

$$(\chi U/Qa)_{Total} = 18.4$$

Table 10

Variation of Normalized Concentration ($\chi U/Qa$, 10^3) with Stability Class and Upwind Extent of Area Source*

Stability Class	r_d to r_u**(km)	0.5-1.0	1.0-2.0	2.0-4.0	4.0-8.0
D		15.74	19.17	24.2	31.5
E		22.5	27.6	34.9	47.1

Source: Ludwig et. al. (1970).

*These normalized concentrations are applicable to mixing depths from 100 m up to approximately 3200 m.

**From the effective distance to the downwind edge of the area source, r_d (km) to the effective distance to the upwind edge of the area source, r_u (km).

WORKSHEET 6--CO AREA SOURCE DISPERSION ANALYSIS
(see instructions following)

Project No.: ____________________ **Analyst:** ____________________

Site: ____________________ **Date:** ____________________

Step	Symbol	Inputs/Units				
		Basic Inputs	Traffic Stream			
1		Source ID				
2	SC	Stability class				
3	U	Wind speed ($m\ s^{-1}$)				
4	σ_{z_o}	Initial dispersion (m)				
5	x_o	Virtual dispersion distance (m)				
6	x_u	Actual upwind distance (m)				
7	$r_u = x_u + x_o$	Effective upwind distance* (m)				
8	x_d	Actual downwind distance (m)				
9	$r_d = x_d + x_o$	Effective downwind distance* (m)				
10	Qa	Emission rate ($g\ m^{-2}\ s^{-1}$)				
		Dispersion Computation				
11	$(\chi U/Qa)_u$	Upwind normalized concentration				
12	$(\chi U/Qa)_s$	Downwind normalized concentration	-	-	-	-
13	$\chi U/Qa$	Normalized CO concentration*				
14	Qa	Emission rate ($g\ m^{-2}\ s^{-1}$)	x	x	x	x
15	χU					
		Enter line 3	÷	÷	÷	÷
16	χ	CO concentration ($mg\ m^{-3}$)				
17	χ	CO concentration (ppm)				

*Use Table 11 to determine $\chi u/Qs$ if $r > 500$ m and skip Steps 11 and 12.

INSTRUCTIONS FOR COMPLETING WORKSHEET 6

Step	Instructions
1	Identify source by ID from Worksheet 3.
2	Determine stability class using Table 7 and enter (use stability E for any F cases).
3	Determine wind speed according to procedures given in Figures 20-23 and enter.
4	Specify $\sigma_{z_0} = 1.5$ m for a smooth surface, or $\sigma_{z_0} = 5.0$ m for a rough surface.
5	Use Table 9 to specify virtual dispersion distance (x_0).
6	Compute x_u as distance from receptor to upwind edge of area source (Figure 15) and enter.
7*	Add line 5 (x_0) and line 6 (x_u) to obtain r_u.
8	Compute x_d as distance upwind from receptor to downwind edge of area source (Figure 15) and enter.
9*	Add line 5(x_0) and line 8 (x_d) to obtain r_d.
10	Enter Qa computed in Worksheet 3.
11	Enter Figure 16 with r_u (line 7) and stability (line 2) to obtain $(\chi U/Qa)_u$.
12	Enter Figure 16 with r_d (line 9) and stability (line 2) to obtain $(\chi U/Qa)_d$.
13*	Subtract line 12 from line 11 to obtain $\chi U/Qa$.
14	Enter Qa (line 10).
15	Multiply lines 12 and 13 to obtain χU.
16	Divide line 15 by line 3 to obtain the computed value of the CO concentration in mg m^{-3}.
17	Multiply line 16 by a conversion factor of .87 to obtain the computed value of the CO concentration in ppm.

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*Use Table 10 to determine $\chi U/Q$ if $r > 500$ m, and skip Steps 11 and 12. See example in text.

E. Determination of Total CO Concentration

1. General

Subsections III-C and III-D discussed methods for determining traffic flow characteristics, CO emission rates, and ambient CO concentrations for three categories of indirect sources: (1) infinite line source, (2) discrete or finite line source, and (3) distributed or area source. The concentration levels predicted by these analyses do not, however, consider the total concentration that is observed at a receptor. The total concentration is also impacted by general background levels at the site. In this section, total CO concentration is determined by considering three aspects: (1) the use of available historical data and CO measurements from a two-week study at the site, (2) the selection of appropriate worst-case meteorological, traffic, and background conditions, and (3) the time under evaluation (i.e., one or eight hours). A total of eight different cases of criteria (1) and (3) can be considered; these are summarized in Table 11, and are discussed at greater length in Subsections 2 (one-hourly) and 3 (eight-hourly).

2. One-Hourly Impact

a. Category 1 (Table 11)

All of the various cases of one-hourly impact analyses are treated under the assumption that CO attributable to the local source is the dominant factor in determining the total one-hourly concentration. Accordingly, the worst-case meteorological, traffic and background conditions are selected as those corresponding to the period of maximum local CO emissions--usually the period of peak traffic demand volume.*

Table 11

SUMMARY OF POSSIBLE DATA ANALYSIS COMBINATIONS

	CO Data		Time Period	
Category	Historical Background	Local Two-Week Monitoring	1-Hourly	8-Hourly
1a	✓	✓	✓	
1b	✓	m	✓	
2	m	✓	✓	
3	m	m	✓	
4a	✓	✓		✓
4b	✓	m		✓
5	m	✓		✓
6	m	m		✓

m = no data available, concentration must be estimated.

Figure 17 summarizes the procedure for analyzing categories 1a and 1b: one-hourly period, historical CO data, and either some or no local aerometric data.

Step 1: Determine the hours(s) of the day and month of the year with peak traffic demand volume; depending on the nature of the source this will usually be the peak sum of source-oriented and other (e.g., through) traffic.

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*Low ambient temperatures affect emission rates especially from cold-starting vehicles, so the user should be aware that lesser than peak volumes of traffic with low temperatures may have higher emissions than highest volumes with high temperatures.

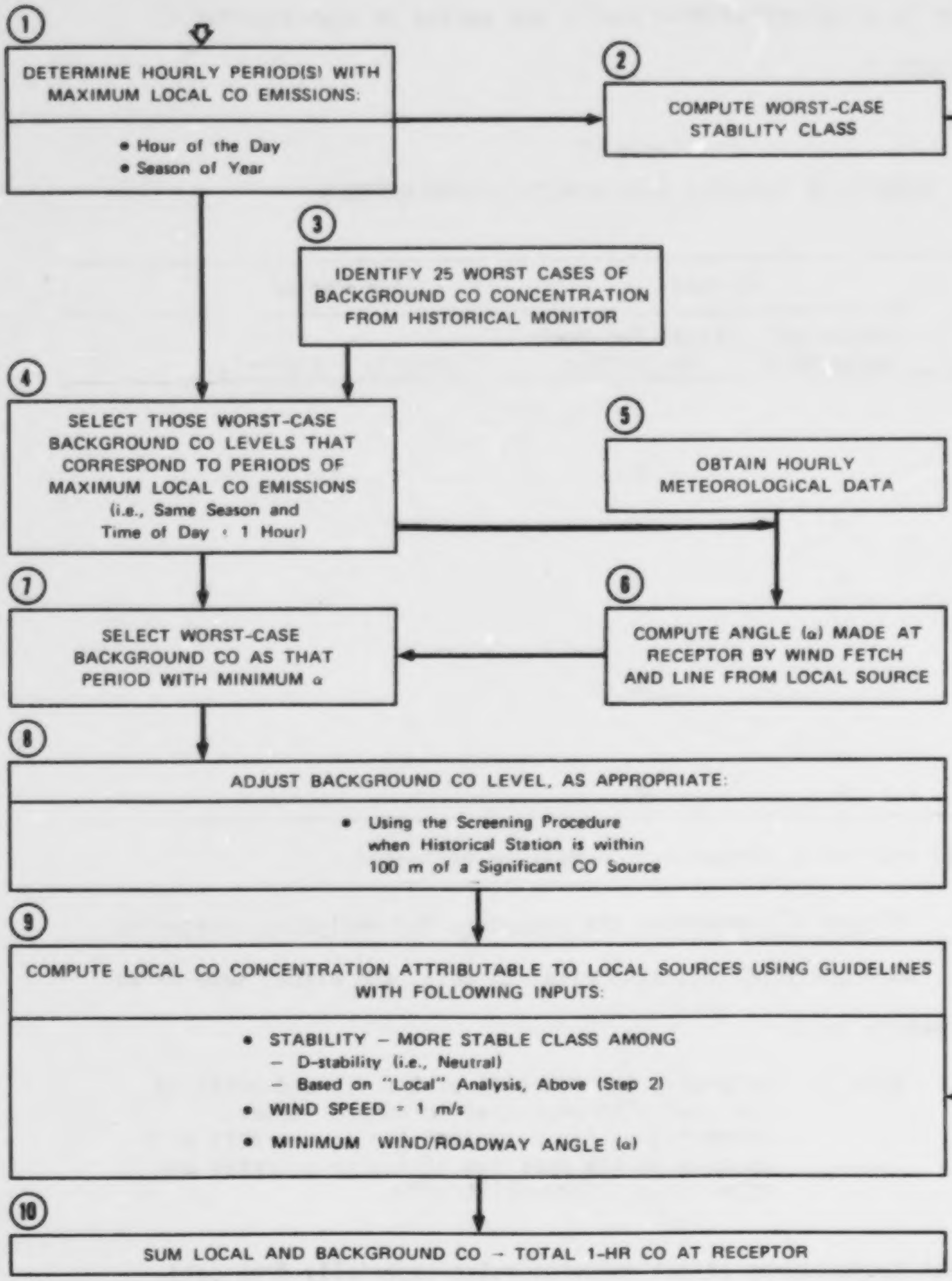

①
DETERMINE HOURLY PERIOD(S) WITH MAXIMUM LOCAL CO EMISSIONS:
• Hour of the Day
• Season of Year
②
COMPUTE WORST-CASE STABILITY CLASS
③
IDENTIFY 25 WORST CASES OF BACKGROUND CO CONCENTRATION FROM HISTORICAL MONITOR
④
SELECT THOSE WORST-CASE BACKGROUND CO LEVELS THAT CORRESPOND TO PERIODS OF MAXIMUM LOCAL CO EMISSIONS
(i.e., Same Season and Time of Day ± 1 Hour)
⑤
OBTAIN HOURLY METEOROLOGICAL DATA
⑦
SELECT WORST-CASE BACKGROUND CO AS THAT PERIOD WITH MINIMUM α
⑥
COMPUTE ANGLE (α) MADE AT RECEPTOR BY WIND FETCH AND LINE FROM LOCAL SOURCE
⑧
ADJUST BACKGROUND CO LEVEL, AS APPROPRIATE:
• Using the Screening Procedure when Historical Station is within 100 m of a Significant CO Source
⑨
COMPUTE LOCAL CO CONCENTRATION ATTRIBUTABLE TO LOCAL SOURCES USING GUIDELINES WITH FOLLOWING INPUTS:
• STABILITY – MORE STABLE CLASS AMONG
– D-stability (i.e., Neutral)
– Based on "Local" Analysis, Above (Step 2)
• WIND SPEED = 1 m/s
• MINIMUM WIND/ROADWAY ANGLE (α)
⑩
SUM LOCAL AND BACKGROUND CO → TOTAL 1-HR CO AT RECEPTOR

Figure 17 Determination of worst-case one-hourly CO impact using historical background CO concentration and indirect-source guidelines, (both with and without data from two-week, local monitoring program).

Step 2: Using Table 7, determine the corresponding worst-case stability category that could occur.

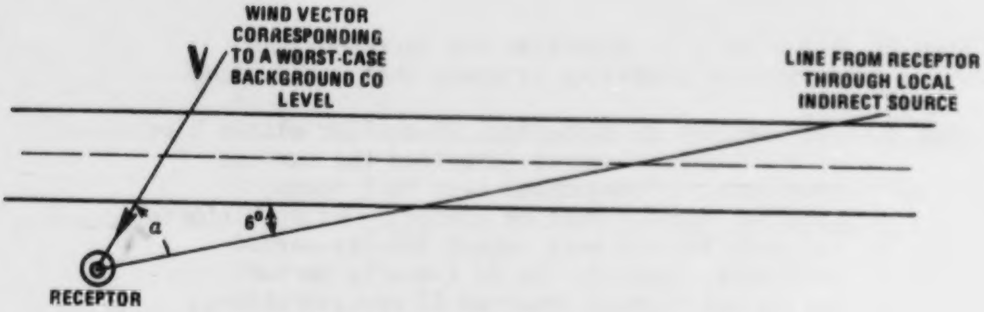
Step 3: (a) Identify an historical CO monitor within 2 km of the indirect source (provided the surface roughness is homogeneous over that range; if not, the monitor must be closer). Using historical data for the most recent 365-day period available, identify the 25 1-hourly periods having the highest observed CO concentrations; note the corresponding hour, month, wind speed, and wind direction. (b) If a local study from a two-week monitoring period is available nearer the indirect source than any historical monitors, this data can be used to identify the highest CO concentrations. However the two-week monitoring period should be from the season identified by an historical monitor as having the worst case CO concentrations. (If not, go to Category 2.) Identify CO measurements for those hours of the two week monitoring study that correspond to periods of maximum local CO emissions.

Step 4: Select those worst-case background CO levels from Step 3 that correspond to the period(s) of maximum local CO emissions identified in Step 1. Corresponding periods should coincide within ± 1 month and ± 1 hour.

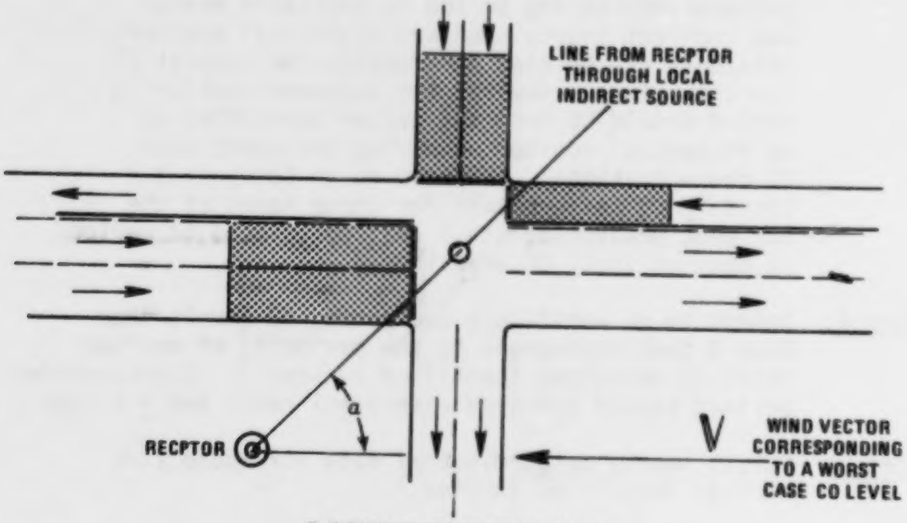
Step 5: Obtain hourly meteorological data for those time periods identified in Step 4.

Step 6: For the worst-case periods identified in Step 4, compute the angle (α) made at the proposed receptor by the intersection of the wind vector and a line drawn from the receptor to the indirect source. Figure 18 illustrates the procedure. (Note that often the indirect source will be of considerable finite length and may have several receptors under evaluation; good judgment must therefore be exercised to determine a representative value of α. The objective is to ascertain a reasonable worst-case background CO at a minimum value of α.)

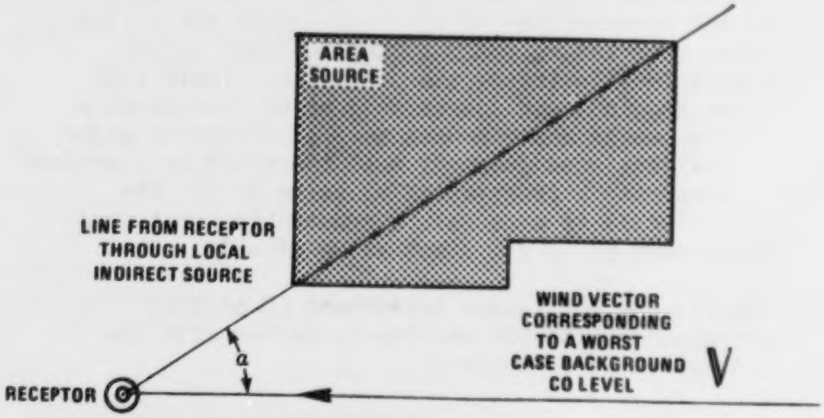
Step 7: Specify the worst-case background CO as that corresponding to the one-hourly period with the minimum value of angle α.


WIND VECTOR CORRESPONDING TO A WORST-CASE BACKGROUND CO LEVEL
V
LINE FROM RECEPTOR THROUGH LOCAL INDIRECT SOURCE
α
6°
RECEPTOR

A. UNINTERRUPTED HIGHWAY/INDIRECT SOURCE


LINE FROM RECPTOR THROUGH LOCAL INDIRECT SOURCE
α
RECPTOR
V
WIND VECTOR CORRESPONDING TO A WORST CASE CO LEVEL

B. INTERSECTION/INDIRECT SOURCE


AREA SOURCE
LINE FROM RECEPTOR THROUGH LOCAL INDIRECT SOURCE
WIND VECTOR CORRESPONDING TO A WORST CASE BACKGROUND CO LEVEL
V
α
RECEPTOR

C. AREA SOURCE/INDIRECT SOURCE

Figure 18 Illustration of the determination of the angle α

Step 8: If either the historical or two-week monitors are within 100 m of any significant CO source, adjust the measured values by subtracting out the local contribution as determined either by the indirect source analysis procedure or locally measured values.

Step 9: Compute the local CO contribution using the indirect-source analysis methodology (III-C, D) with the following inputs: (a) traffic inputs corresponding to data specified in Step 1, and (b) meteorological inputs as follows: (1) atmospheric stability as the output of Step (2); (2) wind speed of 1 m s^{-1}; and (3) wind/roadway angle corresponding to the case of "minimum α."

Step 10: The total one-hourly CO concentration at the specified indirect-source receptor is the sum of the local contribution (Step 9) and the background contribution (Step 8).

b. Category 2 (Table 11)

Category 2 consists of those cases for which it is necessary to compute the one-hourly CO impact when local data are available from a two-week monitoring study but no historical background CO data are available. Figure 19 illustrates the step-by-step procedure for selecting the appropriate hour, using the local CO data, and estimating background to compute the total one-hourly impact.

Step 1: Determine the hour(s) of the day and month of the year with peak traffic demand volume; depending on the nature of the source, this will usually be the peak sum of source-oriented and other (e.g., through) traffic.

Step 2: Using Table 8, determine the corresponding worst-case stability category that could occur.

Step 3: The local, two-week monitoring program ideally should provide data that are representative of both background at the site and the impact of any existing

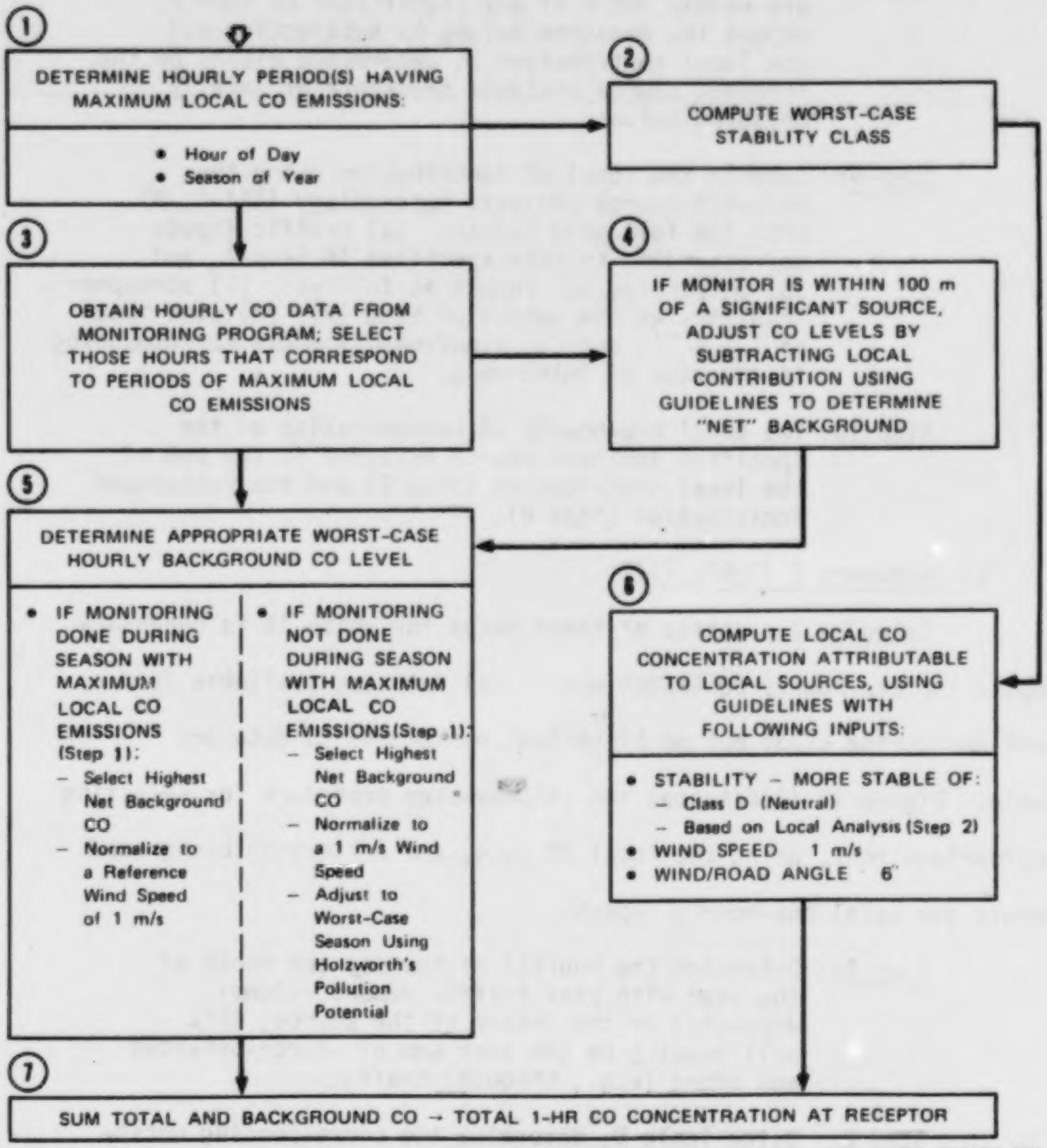

①
DETERMINE HOURLY PERIOD(S) HAVING MAXIMUM LOCAL CO EMISSIONS:
• Hour of Day
• Season of Year
②
COMPUTE WORST-CASE STABILITY CLASS
③
OBTAIN HOURLY CO DATA FROM MONITORING PROGRAM; SELECT THOSE HOURS THAT CORRESPOND TO PERIODS OF MAXIMUM LOCAL CO EMISSIONS
④
IF MONITOR IS WITHIN 100 m OF A SIGNIFICANT SOURCE, ADJUST CO LEVELS BY SUBTRACTING LOCAL CONTRIBUTION USING GUIDELINES TO DETERMINE "NET" BACKGROUND
⑤
DETERMINE APPROPRIATE WORST-CASE HOURLY BACKGROUND CO LEVEL
• IF MONITORING DONE DURING SEASON WITH MAXIMUM LOCAL CO EMISSIONS (Step 1):
– Select Highest Net Background CO
– Normalize to a Reference Wind Speed of 1 m/s
• IF MONITORING NOT DONE DURING SEASON WITH MAXIMUM LOCAL CO EMISSIONS (Step 1):
– Select Highest Net Background CO
– Normalize to a 1 m/s Wind Speed
– Adjust to Worst-Case Season Using Holzworth's Pollution Potential
⑥
COMPUTE LOCAL CO CONCENTRATION ATTRIBUTABLE TO LOCAL SOURCES, USING GUIDELINES WITH FOLLOWING INPUTS:
• STABILITY – MORE STABLE OF:
– Class D (Neutral)
– Based on Local Analysis (Step 2)
• WIND SPEED 1 m/s
• WIND/ROAD ANGLE 6°
⑦
SUM TOTAL AND BACKGROUND CO → TOTAL 1-HR CO CONCENTRATION AT RECEPTOR

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FIGURE 19 DETERMINATION OF WORST-CASE ONE-HOURLY CO IMPACT WHEN ONLY LOCAL DATA ARE AVAILABLE FROM A TWO-WEEK MONITORING PROGRAM

local sources. When both types of measurements are made, Step 4 can be skipped. Also, the "local" CO data can be used to evaluate the performance of the indirect-source review procedure to be followed in Step 6.

Step 4: Identify CO measurements for those hours of the two-week monitoring study that correspond to periods of maximum local CO emissions for the proposed indirect source. If the monitor is within 100 m of any other significant CO source, adjust the measured values by subtracting out the local contribution as determined by the indirect-source analysis procedure. Identify the maximum "net" background so determined and note the corresponding wind speed.

Step 5: (a) Determine the appropriate background CO concentration using the maximum "net" background (Step 4) and normalizing to a reference wind speed of 1 $m\ s^{-1}$[i.e., multiply by the wind speed value ($m\ s^{-1}$) from Step 4]; (b) if the monitoring was not done during the season of maximum local CO emissions (ref: Step 1), then use Holzworth's (1972) method to project the worst-season case. This projection is done by multiplying the background CO from Step 5(a) by the ratio of: (1) the maximum Holzworth CO estimate for the peak season, to (2) the Holzworth CO estimate for the monitoring season.

Step 6: Compute the local CO contribution using the indirect-source analysis methodology (III-C, D) with the following inputs: (2) traffic, Step 1, (b) meteorology, (1) atmospheric stability as the output of Step 2, (2) wind speed of 1 $m\ s^{-1}$, and (3) a specified worst-case wind/roadway angle of 6°; other values may be used if it can be demonstrated that they are more appropriate to worst-case conditions at the site during the time period (i.e., hour) being considered.

Step 7: The total one-hourly CO concentration at the specified indirect-source receptor is the sum of the local contribution (Step 6) and the background contribution (Step 5).

c. Category 3 (Table II)

Category 3 consists of those cases for which it is necessary to compute the one-hourly CO impact when there are no historical or local monitoring data available. Figure 20 illustrates the step-by-step procedure for selecting the appropriate period to analyze and estimating both local and background contributions.

Step 1: Determine the hour(s) of the day and month of the year with peak traffic demand volume; depending on the nature of the source, this will usually be the peak sum of source-oriented and other (e.g., through) traffic.

Step 2: Using Table 8, determine the corresponding worst-case stability category that could possibly occur.

Step 3: Compute the local CO contribution using the indirect-source analysis methodology (III-C, D) with the following inputs: (a) traffic, Step 1; (b) meteorology: (1) atmospheric stability as the output of Step 2, (2) wind speed of 1 $m\ s^{-1}$, (3) a specified worst-case wind/roadway angle of 6°; larger values may be used if it can be demonstrated that they are more appropriate to worst-case conditions at the site during the hour being considered (e.g., at an intersection).

Step 4: Compute the background concentration using the procedure developed by Holzworth (1972)* with the following inputs: (a) time period, hour and season from Step 1, (b) the maximum distance from the receptor to the upwind edge of the city, (c) atmospheric mixing depth, the appropriate minimum values tabulated by Holzworth or other data source, (d) wind speed, as tabulated by

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*Appendix D is an extraction from Holzworth (1972) of the simple urban dispersion model with corresponding seasonal and AM/PM mixing height and wind speed data for 62 sites in the United States.

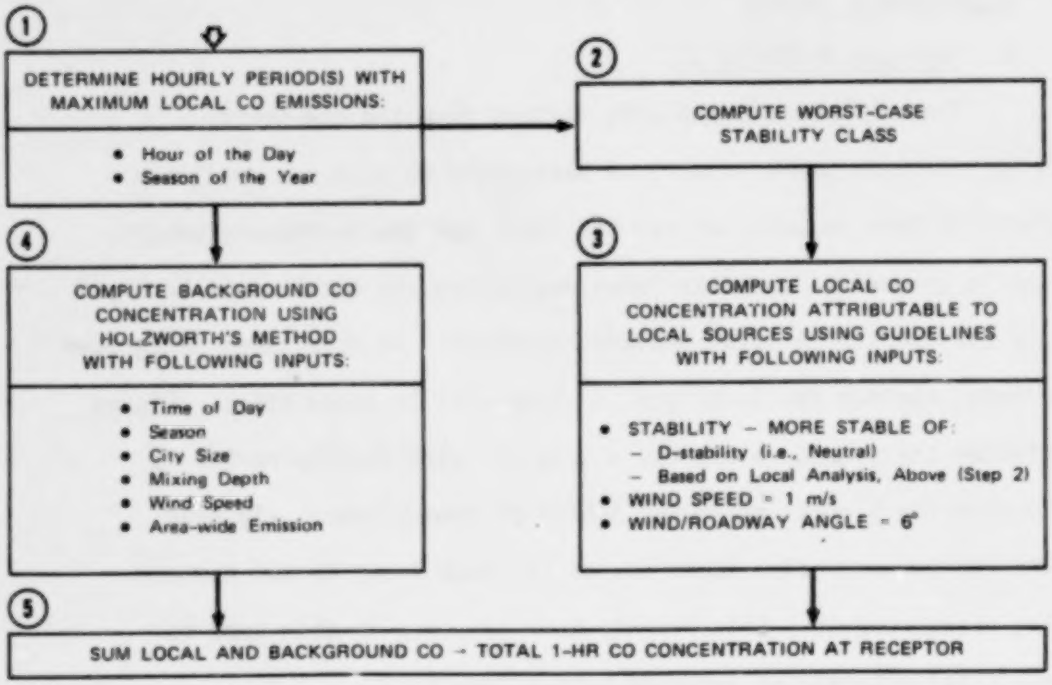

①
DETERMINE HOURLY PERIOD(S) WITH MAXIMUM LOCAL CO EMISSIONS:
• Hour of the Day
• Season of the Year
②
COMPUTE WORST-CASE STABILITY CLASS
③
COMPUTE LOCAL CO CONCENTRATION ATTRIBUTABLE TO LOCAL SOURCES USING GUIDELINES WITH FOLLOWING INPUTS:
• STABILITY – MORE STABLE OF:
– D-stability (i.e., Neutral)
– Based on Local Analysis, Above (Step 2)
• WIND SPEED = 1 m/s
• WIND/ROADWAY ANGLE = 6°
④
COMPUTE BACKGROUND CO CONCENTRATION USING HOLZWORTH'S METHOD WITH FOLLOWING INPUTS:
• Time of Day
• Season
• City Size
• Mixing Depth
• Wind Speed
• Area-wide Emission
⑤
SUM LOCAL AND BACKGROUND CO → TOTAL 1-HR CO CONCENTRATION AT RECEPTOR

Figure 20 Determination of worst-case one-hourly CO impact when no historical or local (i.e., limited) background CO data are available.

Holzworth or other data source, and (3) area-wide emission rates (annual averages are available from EPA (1973)): peak hourly area-wide emission rates may be estimated to be approximately 10% of the daily average.

Step 5: The total one-hourly CO concentration at the specified indirect-source receptor is the sum of the local contribution (Step 3) and the background contribution (Step 4).

3. Eight-Hourly Impact

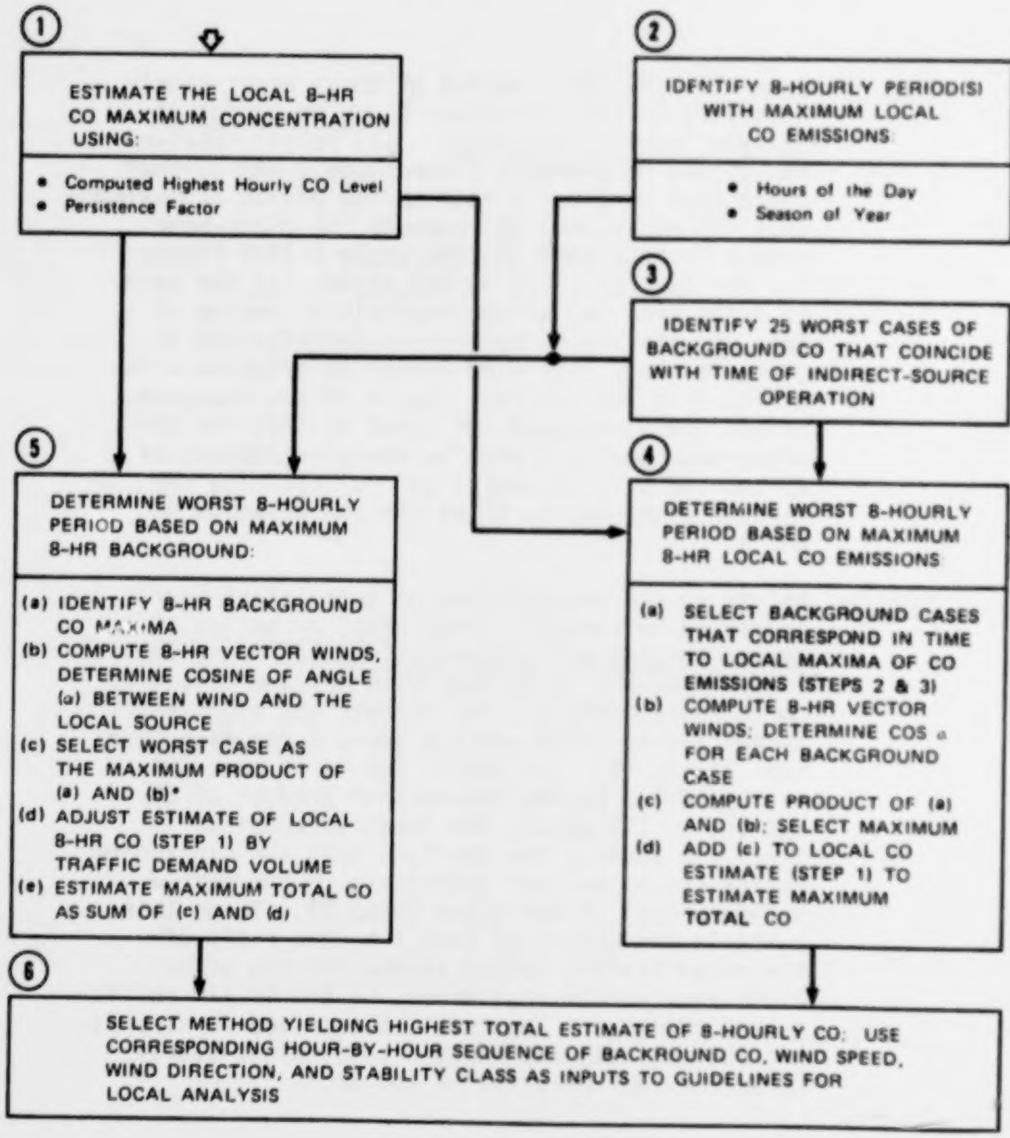
a. Category 4 (Table 11)

The eight-hourly analysis differs from the one-hourly analysis for the case where historical background CO data are available. The analysis differs because neither the local nor the background contribution can be assumed to dominate (when evaluating the total eight-hourly CO average against the 10 mg/m^3 ambient standard). In some cases background will dominate, whereas the local contribution will in other cases. Figure 21 illustrates the procedure whereby a critical eight-hourly period is chosen to give the highest estimated 8-hour CO from either: (1) peak background and corresponding local CO, or (2) peak local CO and the corresponding background CO. This hour-by-hour sequence is then used to generate a final estimate of combined 8-hour impact.

Step 1: Using the maximum one-hourly local CO level computed in the Category 1 procedure (Step 10), estimate the local eight-hourly CO maxima by multiplying by a persistence factor from a representative or local area. (If necessary, a factor of .6-.7 may be used for the persistence factor. This range of factors has resulted from several studies in cities throughout the U.S.)

Step 2: Determine the eight-hourly period and month with peak traffic demand volume.

Step 3: Using data records from a representative historical CO monitor, identify those 25 eight-hourly periods during the past 365 days that have maximum CO averages. These eight-hourly periods should coincide with periods of indirect-source operation.


①
ESTIMATE THE LOCAL 8-HR CO MAXIMUM CONCENTRATION USING:
• Computed Highest Hourly CO Level
• Persistence Factor
②
IDENTIFY 8-HOURLY PERIOD(S) WITH MAXIMUM LOCAL CO EMISSIONS:
• Hours of the Day
• Season of Year
③
IDENTIFY 25 WORST CASES OF BACKGROUND CO THAT COINCIDE WITH TIME OF INDIRECT-SOURCE OPERATION
⑤
DETERMINE WORST 8-HOURLY PERIOD BASED ON MAXIMUM 8-HR BACKGROUND:
(a) IDENTIFY 8-HR BACKGROUND CO MAXIMA
(b) COMPUTE 8-HR VECTOR WINDS, DETERMINE COSINE OF ANGLE (α) BETWEEN WIND AND THE LOCAL SOURCE
(c) SELECT WORST CASE AS THE MAXIMUM PRODUCT OF (a) AND (b)*
(d) ADJUST ESTIMATE OF LOCAL 8-HR CO (STEP 1) BY TRAFFIC DEMAND VOLUME
(e) ESTIMATE MAXIMUM TOTAL CO AS SUM OF (c) AND (d)
④
DETERMINE WORST 8-HOURLY PERIOD BASED ON MAXIMUM 8-HR LOCAL CO EMISSIONS:
(a) SELECT BACKGROUND CASES THAT CORRESPOND IN TIME TO LOCAL MAXIMA OF CO EMISSIONS (STEPS 2 & 3)
(b) COMPUTE 8-HR VECTOR WINDS; DETERMINE COS α FOR EACH BACKGROUND CASE
(c) COMPUTE PRODUCT OF (a) AND (b); SELECT MAXIMUM
(d) ADD (c) TO LOCAL CO ESTIMATE (STEP 1) TO ESTIMATE MAXIMUM TOTAL CO
⑥
SELECT METHOD YIELDING HIGHEST TOTAL ESTIMATE OF 8-HOURLY CO; USE CORRESPONDING HOUR-BY-HOUR SEQUENCE OF BACKROUND CO, WIND SPEED, WIND DIRECTION, AND STABILITY CLASS AS INPUTS TO GUIDELINES FOR LOCAL ANALYSIS

***SEE FOOTNOTE NEXT PAGE**

Figure 21 Determination of worst-case eight-hourly CO impact using historical background CO concentrations. (Both with and without data from two-week local monitoring program).

Step 4: Determine the worst period of total eight-hourly estimated CO concentrations based on period of maximum local CO emissions: (a) select observed background CO averages (from Steps 2 and 3) that correspond within ± 1 hour to the period of maximum local CO emissions; (b) compute the eight-hour vector average wind and the angle α (See Figure 18) for each CO value in (a) above; (c) for each CO value from (a) above, multiply by cosine of α. This adjusts the CO background contribution so that the worst case wind vector is oriented with a line from the indirect source to the receptor. Select the background CO* level as that for the eight-hour period where the computed product of CO and cos α is greatest; and (d) estimate the maximum eight-hourly CO as the sum of Steps 4C and 1.

Step 5: Determine the worst period of total eight-hourly estimated CO concentrations based on period of maximum background concentration; (a) identify those eight-hourly periods with maximum CO concentrations (Step 3); (b) compute the eight-hour vector average wind and the angle α for each case (see Figure 18); (c) select the worst case CO* as the period having the maximum product of CO and cos α; (d) adjust the local CO computed in Step 1 to account for the fact that the eight-hour period chosen may not coincide with the period of maximum local CO emissions (Step 2). To do this, multiply the output of Step 1 by the ratio of: the total traffic demand volume for the actual eight-hour period considered, to the total traffic demand volume for the period identified in Step 2; and (e) estimate the maximum total eight-hourly CO as the sums of Steps 5d and 5c.

---

*When the historical monitor is within 100 m of a local CO source, the indirect source methodology (III-C, D) should be used to adjust the background value from the historical site (a two-week local monitoring program conducted near the CO source may also be used to adjust the background values).

Step 6: Analyze the output of either Step 4 or Step 5 to see which provides a greater estimate of the total eight-hourly CO. Then use the corresponding hour-by-hour sequence of: (a) background CO (Steps 5c or 4c); (b) wind speed, direction, and stability; and (c) traffic conditions to estimate the total 8-hour concentration. Use the procedures given in Section III C-D for each hour and average the 8 1-hour values to obtain an 8-hour concentration.

b. Category 5 (Table 11)

Figure 22 outlines the procedure for computing a representative worst-case eight-hourly CO average when background CO data are available from a two-week local monitoring study, but not from an historical background CO monitor. Since the local monitoring is only for a limited period, the procedure uses Holzworth's (1972) simple model to extrapolate the data to an annual basis.

Step 1: Determine the eight-hourly period (both hours and month) with a peak traffic demand volume.

Step 2: Select those daily eight-hourly periods during the two-week air quality study that correspond to the hours of peak traffic (Step 1); identify the eight-hour period with maximum CO concentration.

Step 3: If the CO monitor is within 100 m of a significant CO emission source, use the analysis procedure (Section III-C, D) to subtract out any local CO contribution and obtain a "net" background CO.

Step 4: Determine the appropriate worst-case, eight-hourly background CO. First, normalize the maximum net background CO (Step 3) to a reference wind speed of 1 m s^{-1}; this is done by multiplying the net background by the corresponding eight-hourly average wind speed (m s^{-1}) as measured during the

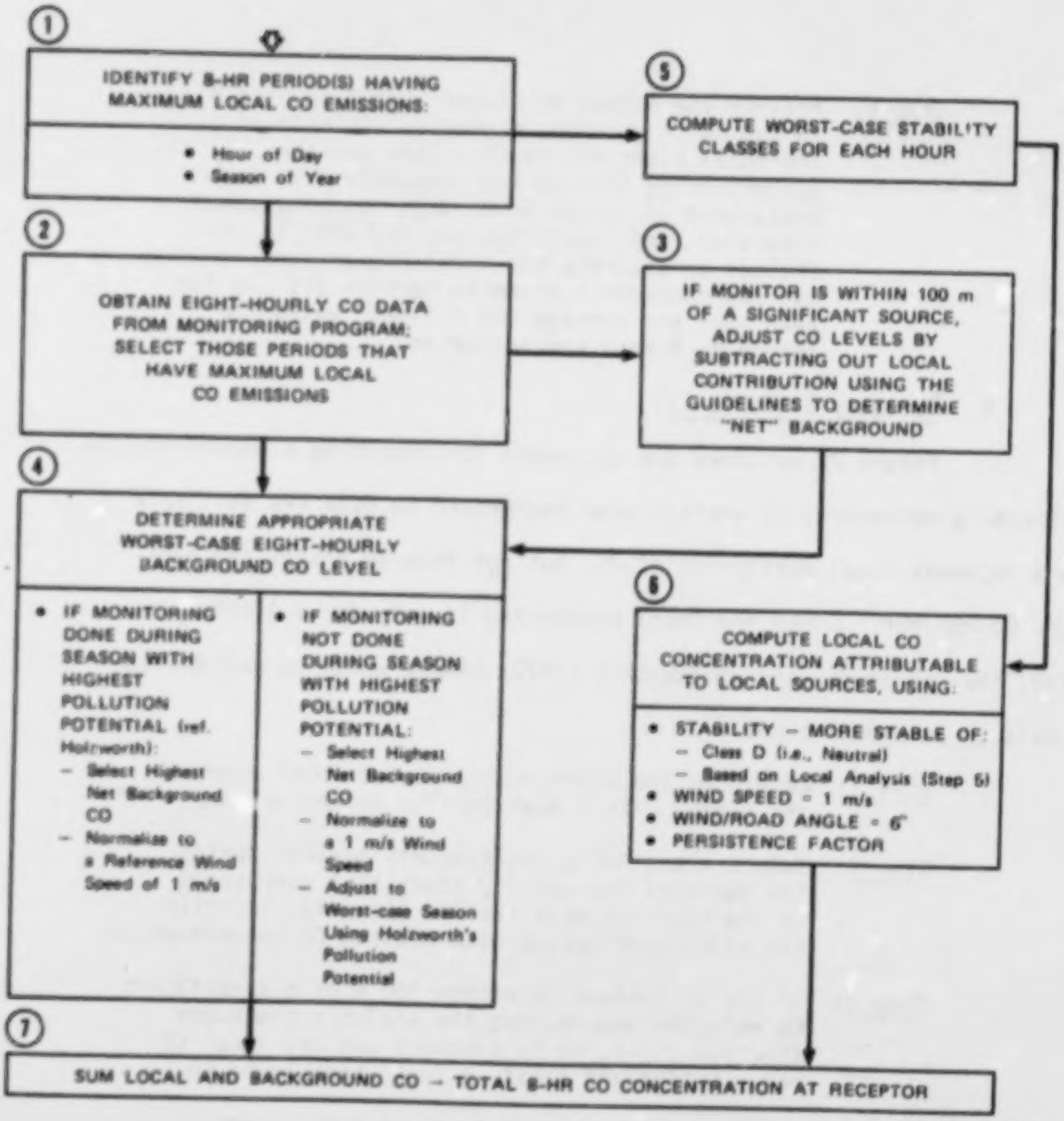

①
IDENTIFY 8-HR PERIOD(S) HAVING MAXIMUM LOCAL CO EMISSIONS:
• Hour of Day
• Season of Year
②
OBTAIN EIGHT-HOURLY CO DATA FROM MONITORING PROGRAM; SELECT THOSE PERIODS THAT HAVE MAXIMUM LOCAL CO EMISSIONS
③
IF MONITOR IS WITHIN 100 m OF A SIGNIFICANT SOURCE, ADJUST CO LEVELS BY SUBTRACTING OUT LOCAL CONTRIBUTION USING THE GUIDELINES TO DETERMINE "NET" BACKGROUND
④
DETERMINE APPROPRIATE WORST-CASE EIGHT-HOURLY BACKGROUND CO LEVEL
• IF MONITORING DONE DURING SEASON WITH HIGHEST POLLUTION POTENTIAL (ref. Holzworth):
– Select Highest Net Background CO
– Normalize to a Reference Wind Speed of 1 m/s
• IF MONITORING NOT DONE DURING SEASON WITH HIGHEST POLLUTION POTENTIAL:
– Select Highest Net Background CO
– Normalize to a 1 m/s Wind Speed
– Adjust to Worst-case Season Using Holzworth's Pollution Potential
⑤
COMPUTE WORST-CASE STABILITY CLASSES FOR EACH HOUR
⑥
COMPUTE LOCAL CO CONCENTRATION ATTRIBUTABLE TO LOCAL SOURCES, USING:
• STABILITY – MORE STABLE OF:
– Class D (i.e., Neutral)
– Based on Local Analysis (Step 5)
• WIND SPEED = 1 m/s
• WIND/ROAD ANGLE = 6°
• PERSISTENCE FACTOR
⑦
SUM LOCAL AND BACKGROUND CO → TOTAL 8-HR CO CONCENTRATION AT RECEPTOR

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FIGURE 22 DETERMINATION OF WORST-CASE EIGHT-HOURLY CO IMPACT WHEN ONLY LOCAL DATA ARE AVAILABLE FROM A TWO-WEEK MONITORING PROGRAM, USING INDIRECT-SOURCE REVIEW GUIDELINES

monitoring study. Consult Holzworth (1972) to determine the season with the peak air pollution potential. If the monitoring was not done during that season, further adjust the net background to an equivalent value for the season of peak pollution potential. Using Holzworth, take the ratio of the worst-case CO concentration for the peak pollution-potential season, to the worst-case CO concentration for the season when the monitoring was done; then multiply the normalized net background by the ratio to obtain an estimate of the worst-case eight-hourly background CO.

Step 5: Determine potential worst-case stability classes (Table 7) for each hour.

Step 6: Using the indirect-source analysis procedure of Section III-C, D, compute the local contribution for the eight-hourly period based on worst-case one-hourly conditions and a persistence factor using: (a) peak one-hourly emission rate; (b) wind speed of 1 m s^{-1}; (c) wind/roadway angle of 6°; and (d) the stability from Step 5. Compute the one-hourly concentration using inputs (a) - (d). Estimate the eight-hourly concentration by multiplying by a persistence factor from a representative or local area. (If necessary, a factor of .6-.7 may be used for the persistence factor).

Step 7: Determine the worst-case eight-hourly average of total CO as the sum of the local CO contribution (Step 6d) and the background CO contribution (Step 4).

c. Category 6 (Table 11)

Figure 23 outlines the procedure for computing a representative worst case total eight-hourly CO average when historical and local background CO data are not available. Since there are no available data in this category, the procedure assumes the joint occurrence of worst-case background and worst-case local contribution.

Step 1: Determine the eight-hourly period (i.e., hours and month) with peak traffic demand volume.

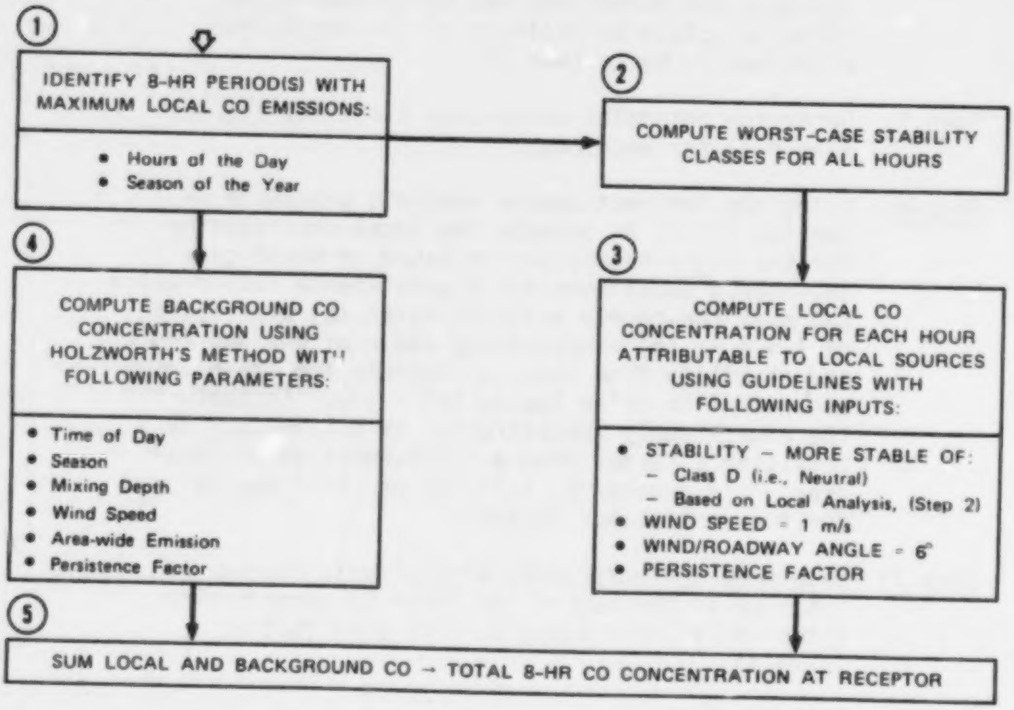
①
IDENTIFY 8-HR PERIOD(S) WITH MAXIMUM LOCAL CO EMISSIONS:
• Hours of the Day
• Season of the Year
②
COMPUTE WORST-CASE STABILITY CLASSES FOR ALL HOURS
④
COMPUTE BACKGROUND CO CONCENTRATION USING HOLZWORTH'S METHOD WITH FOLLOWING PARAMETERS:
• Time of Day
• Season
• Mixing Depth
• Wind Speed
• Area-wide Emission
• Persistence Factor
③
COMPUTE LOCAL CO CONCENTRATION FOR EACH HOUR ATTRIBUTABLE TO LOCAL SOURCES USING GUIDELINES WITH FOLLOWING INPUTS:
• STABILITY – MORE STABLE OF:
– Class D (i.e., Neutral)
– Based on Local Analysis, (Step 2)
• WIND SPEED = 1 m/s
• WIND/ROADWAY ANGLE = 6°
• PERSISTENCE FACTOR
⑤
SUM LOCAL AND BACKGROUND CO → TOTAL 8-HR CO CONCENTRATION AT RECEPTOR

Figure 23 Determination of worst-case eight-hourly CO impact when no historical or local background CO data are available.

Step 2: Determine potential worst-case stability classes for each hour in the eight hourly periods determined in Step 1.

Step 3: Using the indirect-source analysis procedure of Section III-C, D, compute the local CO contribution for the eight-hourly period based on worst-case one-hourly conditions and a persistence factor using: (a) peak one-hourly emission rate; (b) wind speed of 1 m s^{-1}; (c) wind/roadway angle of 6°; and (d) the stability from Step 2. Compute the one-hourly concentration using inputs (a) - (d). Estimate the eight-hourly concentration by multiplying by a persistence factor (a value of 0.6-0.7 may be used unless a more appropriate, locally-derived persistence factor is available).

Step 4: Compute the background CO concentration using the procedure developed by Holzworth (1972) with the following inputs: (a) time period, hour and season from Step 1; (b) city size, the maximum wind fetch over urban and suburban terrain to the receptor location; (c) atmospheric mixing depth, the appropriate minimum values tabulated by Holzworth or other data source; (d) wind speed, as tabulated by Holzworth; (e) area-wide emission rate, annual averages are available from EPA (1973); peak hourly area-wide emissions may be estimated to be approximately 10% of the daily average values; and (f) compute the one-hourly concentration using inputs (a) - (e). Estimate the eight-hourly concentration by multiplying by a persistence factor. (A value of 0.6-0.7 may be used unless a more appropriate, locally derived persistence factor is available).

Step 5: Determine the worst-case eight-hourly average of total CO as the sum of the local CO contribution (Step 3d) and the background CO contribution (Step 4f).

IV. SAMPLE APPLICATIONS

A. Infinite Line Source--Example 1

A receptor point is located 6.1 meters from the north edge of a freeway as shown in Figure 24. From 0800-0900, the wind direction is from 158°, there is neutral stability and smooth terrain, the wind speed is 1.4 m s^{-1}, with air temperature of 70°F and the volume and cruise speed on the freeway are: 2384 veh/hr. and 58 mph, respectively, in the easterly direction of travel, and 4130 veh/hr. and 50 mph, respectively, in the westerly direction of travel. The computations made on the following sample worksheets determine ambient 1-hour local CO concentrations at the specified receptor point.

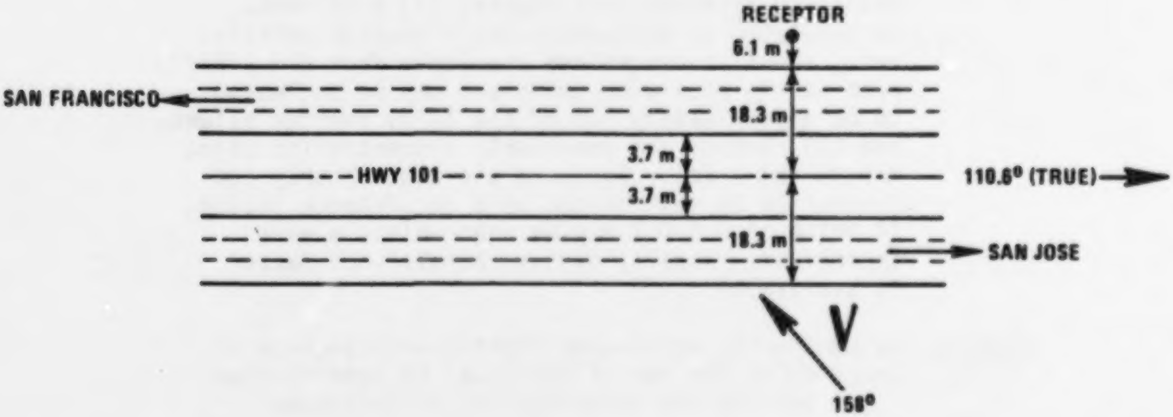

Figure 24 Receptor location for infinite line source

Worksheet 1

TRAFFIC INFORMATION USED IN THE APPLICATION OF THE EVALUATION PROCEDURE

1.	Road segment or intersection approach identification	1E		1W				
2.	Observed 1-hr volume (vph)	2384		4130				
	Observed 8-hr volume (vph)							
	Projected 1-hr peak demand (vph)							
	Projected 8-hr peak demand (vph)							
3.	Percentage cold starts	0		0				
4.	Percentage trucks and buses	5		5				
5.	Metropolitan population							
6.	Slope							
7.	Free-flow parameters							
	Number of lanes	3		3				
	Average lane width (ft)	16		16				
	Design speed (mph)	65		65				
	Highway type (see Figures 2-5)	Fwy		Fwy				
8.	Intersection parameters							
	Intersection designation							
	Approach width (ft)							
	Percentage right turns							
	Percentage left turns							
	Type control and description of signal controller							
9.	Area source parameters							
	Parking lot gate designation							
	Projected 1-hr peak entrance demand (vph)							
	Projected 1-hr peak exit demand (vph)							
	Projected 8-hr peak entrance demand (vph)							
	Projected 8-hr peak exit demand (vph)							
	Parking lot area (m^2)							
	Parking lot capacity (veh)							
	Running time required to access auxiliary parking (s)							
	Facility emptying time							
	Average cars per stall							
	Average area per stall (m^2)							

WORKSHEET 2--LINE SOURCE EMISSION RATE COMPUTATION

(see instructions following)

Project No.: EXAMPLE 2 Analyst: G.J.S.

Site: HWY 101 (CAL.) Date: 6-6-78

Step	Symbol	Input/Units	Traffic Stream			
1	i	Road segment (or approach identification)	1E		1W	
2	V_i	Demand volume (vph)	2384		4130	
3	C_i	Free-flow capacity (vph)				
4	S_i	Cruise speed (mph)	58		50	
5	Ef_i	Free-flow emissions (g/veh-m)	.011		.010	
6.1	N_i	Number of lanes in approach i				
6.2	j	Signalized intersections phase identification				
6.3	$Cs_{i,j}$	Capacity service volume of approach i for phase j (vph of green)				
6.4	$V_{i,j}$	Demand volume for approach i, phase j (vph)				
6.5	C_y	Signal cycle length (s)				
6.6	$G_{i,j}$	Green phase length for approach i, phase j (s)				
6.7	C_i	Capacity of approach i (vph)				
6.8	$P_{i,j}$	Proportion of vehicles that stop				
6.9	$N_{i,j}$	Number of vehicles that stop per signal cycle				
7	N_i	Average number of vehicles in queue at four way stop or two-way stop or end of green phase				
8	Lq_i	Length of vehicle queue for approach i (veh-m/lane)				
9	Rq_i	Average excess running time on approach (s/veh)				
10	Ea_i	Excess emissions from acceleration (g/veh-m)				
11	Ed_i	Excess emissions from deceleration (g/veh-m)				
12	Qad_i	Excess emission rate from acceleration and deceleration (g/m-s)				
13	Lad_i	Length of acceleration and deceleration (m)				
14	Le_i	Length over which excess emissions apply (m)				
15	Fs_i	Average idling emission rate (g/s)				
16	Qe	Average excess emission rate (g/m-s)				
17	Qe'_i	Adjusted excess emission rate (g/s-m)				
18	Qfc_i	Free-flow emission rate (g/s-m)	.008		.013	

$C_T = 1.12$

WORKSHEET 4--INFINITE LINE SOURCE CO DISPERSION ANALYSIS (see instructions following)

Project No.: EXAMPLE 1

Site: HWY 101 (CAL.)

Analyst G.J.S.

Date: 6-6-78

Step	Symbol	Input/Units	Traffic Stream			
		Basic inputs				
		Identification	1E	1W		
1	SC	Stability class				
2	U	Wind speed (m s^{-1})	D	D		
3	θ	Wind/road angle (deg)	1.4	1.4		
4		Sin θ	47	47		
5	x	Road receptor distance (m)	.73	.73		
6	σ_{z_o}	Initial dispersion (m)	35.4	13.4		
7	Q	Emission rate (g m^{-1} s^{-1})	1.5	1.5		
7a		Street canyon? (yes or no)	.008	.013		
			NO			
		Dispersion computation				
8	$\chi U/Q$	Normalized concentration (10^{-3} m^{-1})				
9	χU	Normalized concentration (mg m^{-2} s^{-1})	260	335		
10	χ	CO concentration (mg m^{-3})	2.08	4.36		
11	χ	CO concentration (ppm)	1.49	3.11	=	4.60
			1.29	2.71	=	4.00
		Optional z-correction (heights other than 1.8 m above ground)				
12	z	Height receptor (m)				
13		z-correction factor				
14	χ'	CO concentration at height z (mg/m^3)				
15	χ'	CO concentration at height z (ppm)				

B. Intersection--Example 2

1. Scope

A receptor is located near an intersection as shown in Figure 25.[*] The intersection configuration and signal phasing are also shown on the figure. The signal controller operates on a four-phase cycle, and the first phase allows one of three possible movements depending on the left turn demand. The maximum G/Cy ratio for phase (3) is 0.1. Observed volumes from 1800-1900 are: 1283 on the north approach, 975 on the south approach, 1044 on the east approach, and 458 on the west approach. Left turn demand is 4.5, 8.8, 10, and 10% on the north, south, east, and west approaches, respectively. The wind speed is 3.1 m s^{-1}, the wind direction from 020°, the stability class is neutral (D), the temperature is 70°F, and the terrain is "rough." The following worksheets illustrate the evaluation of the impact of observed traffic on ambient 1-hour local CO concentrations at the specified receptor location (see Figure 25).

2. Guidance (for the Capacity Analysis in Appendix B)

Performing an analysis of capacity for a freeway or an intersection is not a particularly easy task for the technical person lacking traffic engineering experience. Capacity can be determined by proper use of the Highway Capacity Manual (1965) or a conservative estimate of capacity can be determined using Appendix B of these Guidelines. This example makes use of Appendix B and Worksheet B, but some notes on the more complicated entries may prove helpful.

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[*] Note: Figure contains both English and Metric units.

The first entries on Worksheet B are concerned with determining free-flow capacity of either a freeway, urban arterial, or street. In Example 2, the analyst first identified all sections of roadway to be analyzed; for example, 83N designates the north approach of the intersection to be evaluated. Looking at Figure 25, there are two lanes of free-flow traffic approaching from the north, two from the south, two from the east, and one from the west. The maximum free-flow rate of 2000 vehicles per hour must be adjusted for lane widths of less than 12 feet, and for obstructions near the lane edge. In this example, there are no nearby obstructions so it is assumed they are six feet or more away. The appropriate line of Table B-1(c) is then used to determine a lane width correction factor for a non-freeway road. Line 2.2 of Worksheet B shows width adjustment factors for 11 ft. lanes (.88), 12 ft. lanes (1.0), and 10 ft. lanes (.81). No information is given about the percentage of trucks using the road, so an average 5% value is assumed. The capacity during free-flow is determined by applying the Wf and T factors.

Intersection capacity requires the user to determine the signal timing as well as the capacity of each approach per hour of green signal time. The first step in determining intersection capacity is to identify all possible signal phases and the approach to which each applies. Each approach is identified on line 1 of Worksheet B, and two possible control phases may be designated for each approach (line 3.1): the first controls left turning vehicles, the latter controls through traffic. In Example 2, Phases (1a) and

(1b) control left turning traffic on approach 83N: therefore, both are entered on line 3.1 as possible phases. Phase (2) controls through traffic on approach 83N; therefore, it is entered on line 3.1. Similarly, (1a) and (1c) control left turners on 83S. Phase (3) allows left turning vehicles on approach 22E to proceed without interference from opposing traffic; therefore, it is shown in the left turn control column on line 3.1. Phase (4) allows all turning movements, and allows opposing traffic to move at the same time, so it is shown in the through control column on line 3.1. When line 3.1 is completed, all possible signal phases have been identified with some particular movement of traffic.

The approach widths for left and through traffic are shown on line 3.2. Note that if left and through traffic move simultaneously, then the width entered on line 3.2 is the width of the entire approach including left turn lanes.

The percentage of left turners to be entered on line 3.4 must be adjusted if there is no opposing traffic flow. When there is no opposing flow, zero percent is entered for the number of left turners [line 3.4, phases (1a) and (3)]. When there is opposing flow, the percentage of left turners is entered [line 3.4, phase (4)]; when there are no left turns allowed by the signal phase, zero percent of left turners is entered [line 3.4, phase (2)]. Line 3.6 is the capacity service volume of each approach for a particular signal phase as found on Figure B-1. Note that the CBD scale should always be used when determining the capacity of a left turn signal phase.

Demand volume computed on line 4.1 represents the number of vehicles using an approach during a given phase. Note that the demand volume on approach 22E is 1044 vehicles while the capacity service volume is 3200 vehicles per hour. On phase (3) the maximum G/Cy ratio allowed for this approach is 0.1, so the demand is distributed over two phases: 320 for phase (3) (i.e., capacity service volume for 0.1 of a signal cycle), and 724 for phase (4). On line 4.2, a volume to green-capacity ratio is computed and a controlling ratio is identified for each phase (line 4.3). Because phase (2) has the highest V/C allowing all north-south volume demand to pass through in a given cycle, it is appropriate to use phase (1a) as the left turn control phase for north and south. The controlling ratios determine how long each signal phase will last (line 4.6). Once a length for phase (3) is determined, the controlling V/Cs for phase (4) can be determined. In this case the east approach controls phase (4).

Line 4.4 is the sum of the controlling V/Cs ratios and represents a measure of utilization on the entire intersection.

A signal cycle length is computed and entered on line 4.5. In this example, there are four signal phases (1a, 2, 3, 4) but only three amber intervals were given to through traffic. (Most signalized intersections are very simple, but a complex one was chosen here to demonstrate the flexibility of the capacity worksheet.)

Phase length, computed on line 4.6, is usually a minimum of 10 seconds. The phase lengths initially computed on line 4.6 are changed to reflect this minimum. In this example, the green time for phase (1a) was changed from eight seconds to 10 seconds. This increase was accounted for by corresponding decreases of one second for phases (2) and (4). The instructions for line 4.6 indicate that three seconds should be subtracted as the last mathematical step in computing green phase length. This three-second value should not be subtracted from phase (3) because that phase is not terminated by a three-second amber interval. The sum of the green phases for the signal is: 10 + 72 + 20 + 47 = 149; the sum of the amber intervals is: 3 + 3 + 0 + 3 = 9; and the cycle length of the signal is sum of the green and amber intervals: 149 + 9 = 158 which agrees with line 4.5.

Note that an analyst may avoid use of the capacity appendix and worksheet if he can determine approach capacities and signal timing using an alternate source. The traffic engineer for a local jurisdiction will usually know these values for existing intersections. Also, those values would usually be more representative than the values found using Appendix B. When a new intersection is being built, the design engineer will usually have computed a theoretical capacity and signal timing, and this also would be more representative than the values found using Appendix B.

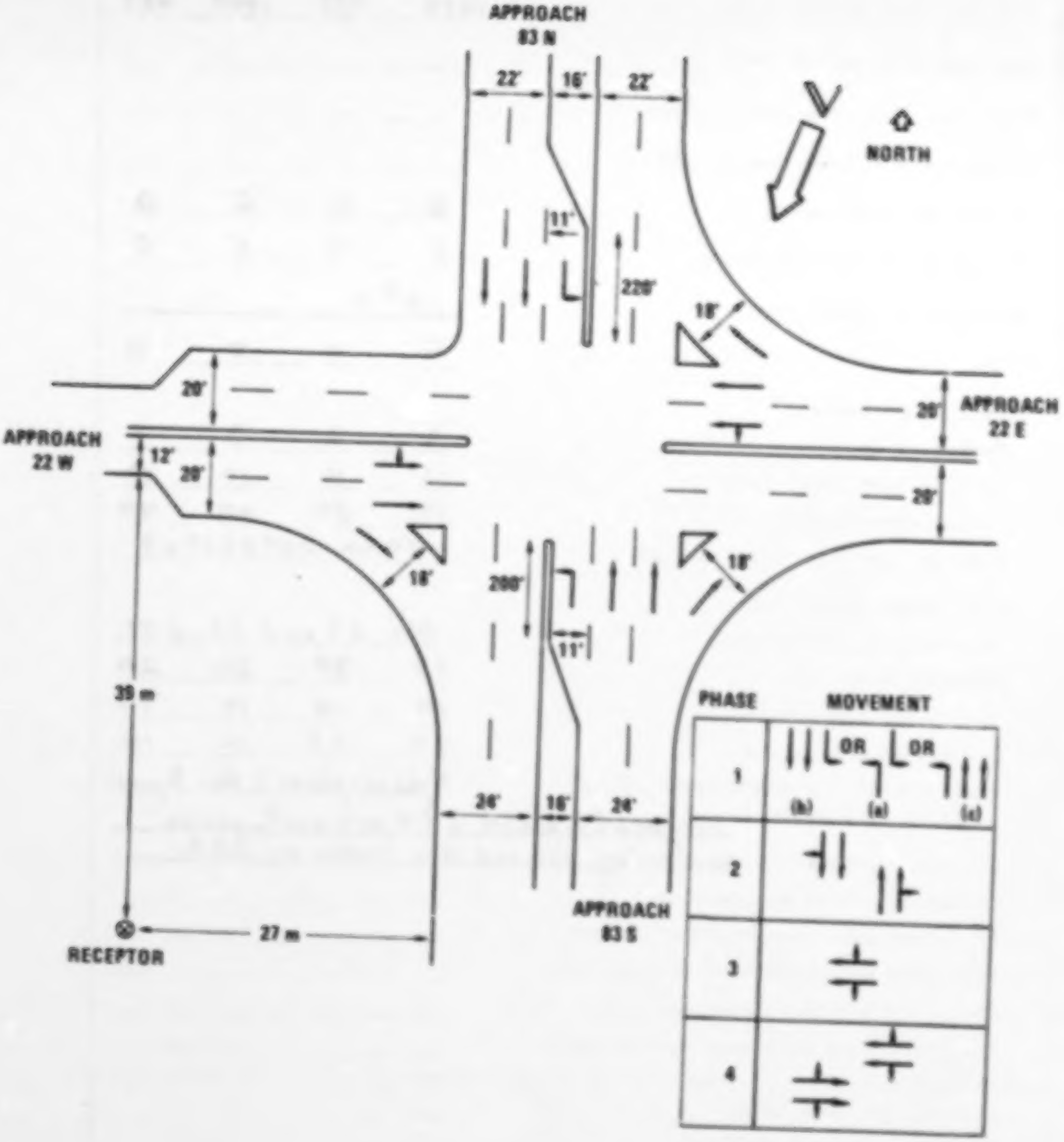
APPROACH 83 N
22'
16'
22'
11'
220'
18'
NORTH
20'
APPROACH 22 E
20'
APPROACH 22 W
12'
20'
20'
18'
200'
18'
11'
39 m
24'
16'
24'
APPROACH 83 S
RECEPTOR
27 m
PHASE
MOVEMENT
1
OR
OR
(b)
(a)
(c)
2
3
4

Figure 25. Receptor location at an intersection.

Worksheet 1

TRAFFIC INFORMATION USED IN THE APPLICATION OF THE EVALUATION PROCEDURE

1.	Road segment or intersection approach identification	83N	83S	22E	22W
2.	Observed 1-hr volume (vph)	1283	975	1044	458
	Observed 8-hr volume (vph)				
	Projected 1-hr peak demand (vph)				
	Projected 8-hr peak demand (vph)				
3.	Percentage cold starts	0	0	0	0
4.	Percentage trucks and buses	5	5	5	5
5.	Metropolitan population	10^6+			
6.	Slope	0	0	0	0
7.	Free-flow parameters				
	Number of lanes	2	2	2	1
	Average lane width (ft)	12	11	10	12
	Design speed (mph)	50	50	40	40
	Highway type (see Figures 2-5)	URBAN ARTERIALS			
8.	Intersection parameters				
	Intersection designation	Rte. 83 and 22nd St.			
	Approach width (ft)	33	35	20	20
	Percentage right turns	10	10	10	10
	Percentage left turns	4.5	8.8	10	10
	Type control and description of signal controller	4 phase: phase 1 has 3 poss. movements; phases 3 & 4 are mult. phases controlling left and thru traffic on 22E.			
9.	Area source parameters				
	Parking lot gate designation				
	Projected 1-hr peak entrance demand (vph)				
	Projected 1-hr peak exit demand (vph)				
	Projected 8-hr peak entrance demand (vph)				
	Projected 8-hr peak exit demand (vph)				
	Parking lot area (m^2)				
	Parking lot capacity (veh)				
	Running time required to access auxiliary parking (s)				
	Facility emptying time				
	Average cars per stall				
	Average area per stall (m^2)				

WORKSHEET B--CAPACITY ANALYSIS (see instructions following)

Step	Symbol	Input/Units	
1	i	Road segment (or approch) designation	83N 83S 22E 22W
2		Free flow capacity computation:	
2.1	M_i	Number of lanes	2 2 2 1
2.2	W_f	Adjustment for lane width (Table B-1)	0.88 1.00 0.81 1.00
2.3	T_i	Adjustment for trucks (Table B-2)	0.95 0.95 0.95 0.95
2.4	C_i	Free flow capacity	3340 3800 3080 1900
3		Signalized intersection capacity:	
3.1	j	Green signal phase identification	[illegible] ② 1c ⓵ₐ 2 ③ ④ _ 4
3.2	Wa_i	Approach width with parking (ft)	12/41 30 12/45 72 28 28 _ 28
3.3		Percent right turners	0 10 0 10 10 10 _ 10
3.4		Percent left turners	0 0 0 0 0 10 _ 10
3.5		Metropolitan area size	
3.6	$Cs_{i,j}$	Capacity service volume (vph of green)	1900/4400 3200 1900/4700 3400 3200 2800 _ 2800
4		Signalized intersection green phase and cycle length:	
4.1	$V_{i,j}$	Demand volume for approach and phase	[illegible] 1225 [illegible] 896 320 724 _ 458
4.2	$V_{i,j}/Cs_{i,j}$	Volume to green capacity ratio	[illegible] [illegible] [illegible] [illegible] [illegible] [illegible] _ [illegible]
4.3	approx G/Cy	Approximate G/Cy	_ .383 .456 _ .100 .257 _ _
4.4	$\sum_j \max(V_{i,j}/Cs_{i,j})$	Sum of the maximum V/C ratios for each signal phase	.798
4.5	Cy	Signal cycle time (sec)	158
4.6	Gj	Green phase length	10 72 10 72 20 47 _ 47
4.7	Gj/Cy	Green phase to cycle time ratio	.063 .456 .063 .456 .127 .297 _ .297
4.8	$C_{i,j}$	Capacity for approach i phase i	88 [illegible] 88 [illegible] [illegible] [illegible] _ [illegible]
5		Two-way stop, two-way yield or uncontrolled intersection:	
5.1	$V_m + V_n$	Major street two-way volume	
5.2	C_i	Cross street capacity	
6		Four-way stop intersections:	
6.1	V_i	Approach volume	
6.2	Spi	Demand split on cross streets	
6.3	C_i	Capacity of approach	
7	C_i	Approach capacity $\sum_j C_{i,j}$ 5.3 for a four-way stop or 6.2 for a two-way stop	1547 1638 1238 832

WORKSHEET 2--LINE SOURCE EMISSION RATE COMPUTATION
(see instructions following)

Project No.: EXAMPLE 2 Analyst: GJS

Site: Rt 83 & 22nd St. Date: 6-6-78

Step	Symbol	Input/Units	Traffic Stream							
1	i	Road segment (or approach identification)	83N		83S		22E		22W	
2	V_i	Demand volume (vph)	1283		975		1044		458	
3	C_i	Free-flow capacity (vph)	3340		3800		3080		1900	
4	S_i	Cruise speed (mph)	46		47		37		37	
5	Ef_i	Free-flow emissions (g/veh-m)	.010		.010		.011		.011	
6.1	N_i	Number of lanes in approach i	3		3		2		2	
6.2	j	Signalized intersections phase identification	1a	2	1a	2	3	4	—	4
6.3	$Cs_{i,j}$	Capacity service volume of approach i for phase j (vph of green)	1400	3200	1400	3400	3300	2800	—	2800
6.4	$V_{i,j}$	Demand volume for approach i, phase j (vph)	58	1225	79	876	320	724	—	458
6.5	C_y	Signal cycle length (s)	158							
6.6	$G_{i,j}$	Green phase length for approach i, phase j (s)	10	72	10	72	20	47	—	47
6.7	C_i	Capacity of approach i (vph)	1547		1638		1238		832	
6.8	$P_{i,j}$	Proportion of vehicles that stop	.98	.88	.99	.73	.97	.95	—	.84
6.9	$N_{i,j}$	Number of vehicles that stop per signal cycle	2.5	47.3	3.4	28.1	13.6	30.2	—	16.7
7	N_i	Average number of vehicles in queue at four way stop or two-way stop or end of green phase	4.86		1.47		5.38		1.22	
8	Lq_i	Length of vehicle queue for approach i (veh-m/lane)	146		88		197		72	
9	Rq_i	Average excess running time on approach (s/veh)	51		37		73		52	
10	Ea_i	Excess emissions from acceleration (g/veh-m)	.080		.080		.082		.082	
11	Ed_i	Excess emissions from deceleration (g/veh-m)	.014		.014		.016		.016	
12	Qad_i	Excess emission rate from acceleration and deceleration (g/m-s)	.030		.019		.027		.011	
13	Lad_i	Length of acceleration and deceleration (m)	189		197		122		122	
14	Le_i	Length over which excess emissions apply (m)	146		88		197		72	
15	Fs_i	Average idling emission rate (g/s)	6.26		3.14		7.99		2.38	
16	Qe	Average excess emission rate (g/m-s)	.082		.078		.057		.052	
17	Qe'_i	Adjusted excess emission rate (g/s-m)	.092		.087		.064		.058	
18	Qfc_i	Free-flow emission rate (g/s-m)	.004		.003		.004		.002	

WORKSHEET 5 - INTERSECTION CO DISPERSION ANALYSIS

(see instructions following)

PROJECT NO.: EXAMPLE 2

ANALYST: GJS

SITE: Rt.83 & 22nd St.

DATE: 6-6-78

LINE NO.	SYMBOL	INPUT/UNITS	TRAFFIC STREAM			
		BASIC INPUTS	83N	83 S	22 E	22W
1	SC	STABILITY CLASS	D	D	D	D
2	U	WIND SPEED (m s^{-1})	3.1	3.1	3.1	3.1
3	θ	WIND-ROAD ANGLE (deg)	20	20	70	70
4	x	LATERAL DISTANCE (m)	30	43	46	41
5	Yu	MAXIMUM LONGITUDINAL DISTANCE (m)	215	56	264	47
6	Yd	MINIMUM LONGITUDINAL DISTANCE (m)	69	0	67	0
7	σ_{z_0}	INITIAL DISPERSION (m)	5	5	5	5
8	Qe	EXCESS EMISSIONS RATE (g m^{-1} s^{-1})	.092	.087	.064	.058
9	Qf	FREE FLOW EMISSIONS RATE (g m^{-1} s^{-1})	.004	.003	.004	.002
9a		STREET CANYON? YES OR NO	NO	NO	NO	NO
		DISPERSION ANALYSIS				
10	χUQ^{-1}	NORMALIZED CONCENTRATION (10^{-3} m^{-1}) FREE FLOW	280	240	120	130
	Qf	ENTER LINE 9	× .004	× .003	× .004	× .002
11	χU	NORMALIZED CONCENTRATION (mg m^{-2} s^{-1})	1.12	.72	.48	.26
	U	ENTER LINE 2	÷ 3.1	÷ 3.1	÷ 3.1	÷ 3.1
12	χ	CO CONCENTRATION (mg m^{-3}) THROUGH EMISSIONS	.36	.23	.15	.08
13	χUQ^{-1}	NORMALIZED CONCENTRATION (FOR Yu)	230	0	120	100
	Qe	ENTER LINE 8	.092	.087	.064	.058
14	χU	NORMALIZED CONCENTRATION (mg m^{-2} s^{-1})	21.16	0	7.68	5.80
	U	ENTER LINE 2	÷ 3.1	÷ 3.1	÷ 3.1	÷ 3.1
15	χ	CO CONCENTRATION--"MAXIMUM QUEUE"	6.83	—	2.48	1.87
16	χUQ^{-1}	NORMALIZED CONCENTRATION (FOR Yd)	30	0	110	0
	Qe	ENTER LINE 8	× .092	× .087	× .064	× .058
17	χU	NORMALIZED CONCENTRATION (mg m^{-1} s^{-1})	2.76	0	7.04	0
	U	ENTER LINE 2	÷ 3.1	÷ 3.1	÷ 3.1	÷ 3.1
18	χ	CO CONCENTRATION--"IMAGINARY QUEUE"	.89	0	2.27	0
19	χ	CO (mg m^{-3}) TOTAL	6.30	.23	.36	1.95 =8.84
20	χ	CO CONCENTRATION (ppm)--TOTAL	5.48	.20	.31	1.70 =7.69
		OPTIONAL z-CORRECTION (HEIGHTS OTHER THAN 1.8 m ABOVE THE GROUND)				
21	z	HEIGHT OF RECEPTOR (m)				
22		z CORRECTION FACTOR				
23	χ'	CO CONCENTRATION AT HEIGHT z (mg/m^3)				
24	χ'	CO CONCENTRATION AT HEIGHT z (ppm)				

C. Area Source--Example 3

1. Scope

A receptor is located in a parking lot of a shopping center as shown in Figure 26. From 1000-1100, the entrance and exit volumes are as shown on the figure. The arrivals are all assumed to be hot start vehicles while the departures are considered to be cold start vehicles. An average base running (Brt) of 270 seconds was determined by an on-site experiment. Each gate to the center functions as if controlled by a vehicle actuated signal which equally splits green time between two phases. The number of parking spaces occupied is 1600. The parking lot area is 234,000 m^2 and the parking capacity is 6000 veh; the terrain is rough and at high altitude. Conditions during the hour are: neutral stability, wind speed 1.79 m s^{-1}, wind direction from 075°, and temperature 70°F. The ambient 1-hour local CO concentration at the receptor point specified in the fiture is estimated in the following worksheets.

2. Guidance

Figure 26 is not detailed enough to get such information as approach width and turning lanes. The approach widths were determined with the aid of a large, scaled engineering drawing and entered in Worksheet 1. Capacities were determined for each entrance and exit gate using Worksheet B. The entrance and exit capacities were considered equal because each entrance-exit pair has a similar lane configuration.

The capacity service volumes were found using the non-CBD scale on Figure B-1. Usually the capacity for turning vehicles would be found using the CBD scale for left turning vehicles, but in this case, right turns are permitted on a red signal indication. As a result, additional capacity must be allowed for each intersection approach so the non-CBD scale was used. Gate S11 is an exception since it has only one exit lane for left and right turners, and the left turners limit the movement of right-turning vehicles on red; therefore, the CBD scale was used to determine capacity service volume at this gate. The capacities are entered directly on Worksheet 3 from Worksheet B without using Worksheet 2 as intermediate step.

The average emission factor is adjusted for all gates using a percent cold start equal to the sum of all exit volumes divided by the total of entering and exiting volumes.

The dispersion analysis on Worksheet 6 has been divided into two parts to account for the emissions void created by the location of the shopping mall upwind of the receptor.

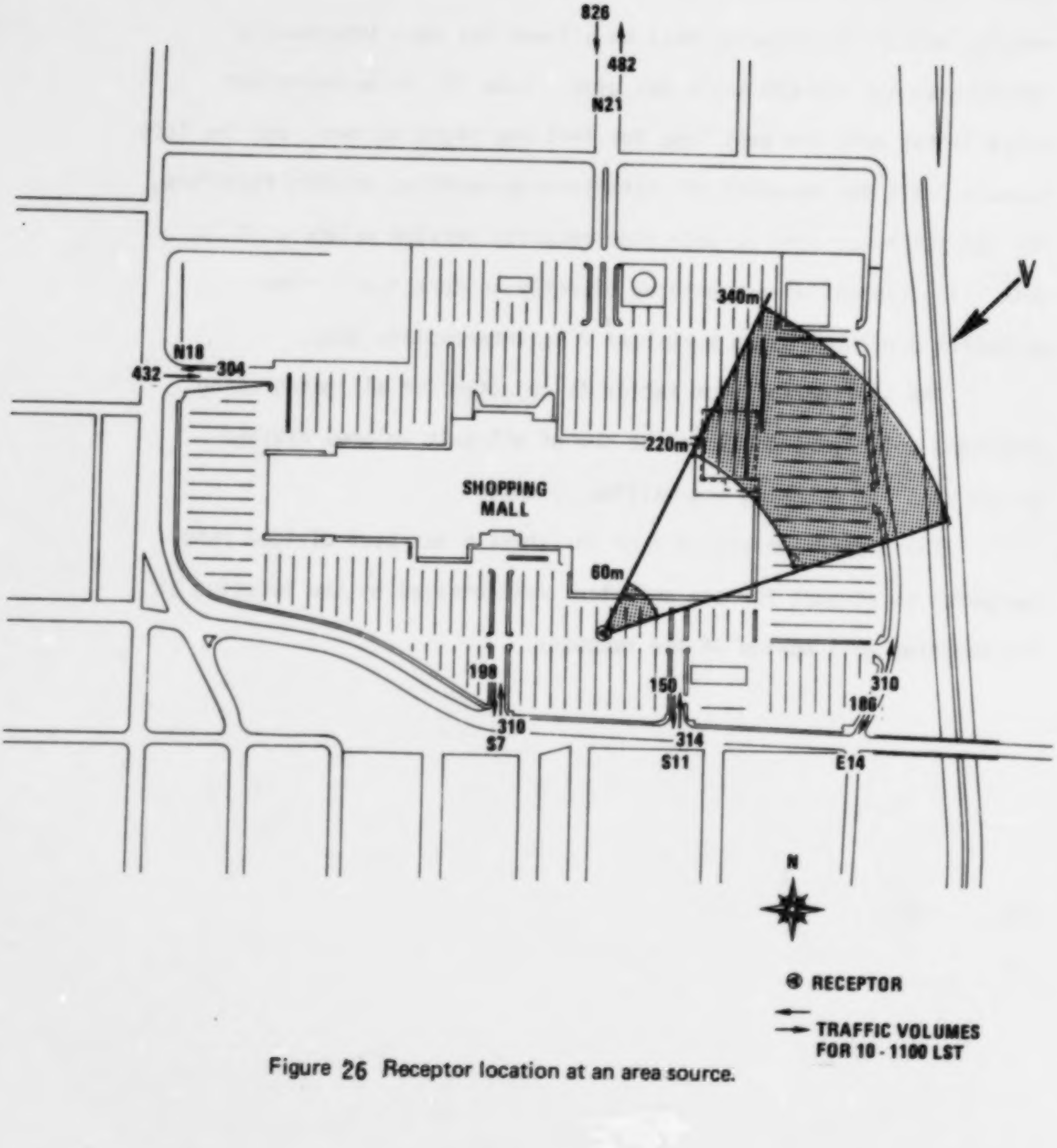
826
482
N21
N18
432
304
340m
V
220m
SHOPPING MALL
60m
198
310
S7
150
314
S11
310
186
E14
N
RECEPTOR
TRAFFIC VOLUMES FOR 10 - 1100 LST

Figure 26 Receptor location at an area source.

Worksheet 1

TRAFFIC INFORMATION USED IN THE APPLICATION OF THE EVALUATION PROCEDURE

1.	Road segment or intersection approach identification	N21 E14 S11 S7 N18
2.	Observed 1-hr volume (vph)	
	Observed 8-hr volume (vph)	
	Projected 1-hr peak demand (vph)	
	Projected 8-hr peak demand (vph)	
3.	Percentage cold starts	
4.	Percentage trucks and buses	
5.	Metropolitan population	5×10^5
6.	Slope	
7.	Free-flow parameters	
	Number of lanes	
	Average lane width (ft)	
	Design speed (mph)	
	Highway type (see Figures 2-5)	
8.	Intersection parameters	
	Intersection designation	N21 E14 S11 S7 N18
	Approach width (ft)	32 32 20 32 28
	Percentage right turns	0 0 0 0 0
	Percentage left turns	0 0 0 0 0
	Type control and description of signal controller	V-A, 60 sec cycle 30 sec per phase
9.	Area source parameters	
	Parking lot gate designation	N21 E14 S11 S7 N18
	Projected 1-hr peak entrance demand (vph)	826 310 314 310 432
	Projected 1-hr peak exit demand (vph)	482 186 150 198 304
	Projected 8-hr peak entrance demand (vph)	
	Projected 8-hr peak exit demand (vph)	
	Parking lot area (m^2)	$2.34 \times 10^5 m^2$
	Parking lot capacity (veh)	6000
	Running time required to access auxiliary parking (s)	
	Facility emptying time	
	Average cars per stall	
	Average area per stall (m^2)	1

WORKSHEET B--CAPACITY ANALYSIS (see instructions following)

Step	Symbol	Input/Units				
1	i	Road segment (or approch) designation	N21	E14	S11	S7
2		Free flow capacity computation:				
2.1	M_i	Number of lanes				
2.2	W_i	Adjustment for lane width (Table B-1)				
2.3	T_i	Adjustment for trucks (Table B-2)				
2.4	C_i	Free flow capacity				
3		Signalized intersection capacity:				
3.1	j	Green signal phase identification				
3.2	Wa_i	Approach width with parking (ft)	32	32	20	32
3.3		Percent right turners	0	0	0	0
3.4		Percent left turners	0	0	0	0
3.5		Metropolitan area size	5×10^5			
3.6	$Cs_{i,j}$	Capacity service volume (vph of green)	3000		1500	
4		Signalized intersection green phase and cycle length:				
4.1	$V_{i,j}$	Demand volume for approach and phase				
4.2	$V_{i,j}/Cs_{i,j}$	Volume to green capacity ratio				
4.3	approx G/Cy	Approximate G/Cy				
4.4	$\sum_j \max(V_{i,j}/Cs_{i,j})$	Sum of the maximum V/C ratios for each signal phase				
4.5	Cy	Signal cycle time (sec)	60			
4.6	Gj	Green phase length	27	27	27	27
4.7	Gj/Cy	Green phase to cycle time ratio	.45	.45	.45	.45
4.8	$C_{i,j}$	Capacity for approach i phase i	1350	1350	675	1350
5		Two-way stop, two-way yield or uncontrolled intersection:				
5.1	$V_m + V_n$	Major street two-way volume				
5.2	C_i	Cross street capacity				
6		Four-way stop intersections:				
6.1	V_i	Approach volume				
6.2	Spi	Demand split on cross streets				
6.3	C_i	Capacity of approach				
7	C_i	Approach capacity $\sum_j C_{i,j}$ 5.3 for a four-way stop or 6.2 for a two-way stop	1350	1350	675	1350

WORKSHEET B--CAPACITY ANALYSIS (see instructions following)

Step	Symbol	Input/Units	
1	i	Road segment (or approch) designation	N18
2		Free flow capacity computation:	
2.1	M_i	Number of lanes	
2.2	W_i	Adjustment for lane width (Table B-1)	
2.3	T_i	Adjustment for trucks (Table B-2)	
2.4	C_i	Free flow capacity	
3		Signalized intersection capacity:	
3.1	j	Green signal phase identification	
3.2	Wa_i	Approach width with parking (ft)	28
3.3		Percent right turners	
3.4		Percent left turners	
3.5		Metropolitan area size	
3.6	$Cs_{i,j}$	Capacity service volume (vph of green)	2660
4		Signalized intersection green phase and cycle length:	
4.1	$V_{i,j}$	Demand volume for approach and phase	
4.2	$V_{i,j}/Cs_{i,j}$	Volume to green capacity ratio	
4.3	approx G/Cy	Approximate G/Cy	
4.4	$\sum_j \max(V_{i,j}/Cs_{i,j})$	Sum of the maximum V/C ratios for each signal phase	
4.5	Cy	Signal cycle time (sec)	60
4.6	Gj	Green phase length	27
4.7	Gj/Cy	Green phase to cycle time ratio	.45
4.8	$C_{i,j}$	Capacity for approach i phase i	1200
5		Two-way stop, two-way yield or uncontrolled intersection:	
5.1	$V_m + V_n$	Major street two-way volume	
5.2	C_i	Cross street capacity	
6		Four-way stop intersections:	
6.1	V_i	Approach volume	
6.2	Spi	Demand split on cross streets	
6.3	C_i	Capacity of approach	
7	C_i	Approach capacity $\sum_j C_{i,j}$ 5.3 for a four-way stop or 6.2 for a two-way stop	1200

WORKSHEET 3--AREA SOURCE EMISSIONS COMPUTATION
(see instructions following)

Project No.: EXAMPLE 3
Analyst: GJS
Site: Shopping MALL
Date: 6-6-78

Step	Symbol	Input/Units				
1	Brt	Base running time				
1.1		Base approach time(s)				
1.2		Base entrance time(s)				
1.3		Base movement-in time(s)				
1.4		Base stop, base start time(s)				
1.5		Base movement-out time(s)				
1.6		Base exit time(s)				
1.7		Base departure time(s)				
1.8		Total base running time(s)	270			
2	A	Area of parking lot (m^2)	2.34×10^5			
3	i	Entrance approach identification	N21	E14	S11	S7
4	Ve_i	Entrance demand volume (vph)	826	310	314	310
5	Ce_i	Entrance approach capacities (vph)	1350	1350	675	1350
6	i	Exit approach identification	N21	E14	S11	S7
7	Vx_i	Exit demand volume (vph)	482	186	150	198
8	Cx_i	Exit approach capacities (vph)	1350	1350	675	1350
9		Number of parking spaces occupied	1600		COLD STARTS 37%	
10	F	Emissions	.63	.63	.63	.63 $C_T = 1.50$
11	Pc	Capacity of parking lot (veh)	6000			
12	Rmi	Excess movement-in time(s)				
13	Fet	Facility emptying time(s)				
14		Excess running time				
14.1	Ve_i/Ce_i	Entering volume-to-capacity ratio	.61	.23	.47	.23
14.2	Vx_i/Cx_i	Exiting volume-to-capacity ratio	.36	.14	.22	.15
14.3	Re_i	Excess running time entering parking lot	4.2	.8	4.6	.8
14.4	Rx_i	Excess running time exiting parking lot	1.4	.4	1.5	.5
15	Te_i	Total entering running time (s/veh)	139	136	140	136
16	Rmo	Excess running time moving out of parking stalls (s/veh)	0			
17	Tx_i	Total exiting running time (s/veh)	136	135	137	135
18	Q_a	Total emission rate from a parking lot ($g/m^2 - s$)	.00036			
19	Q_a'	Area source emission rate without the emissions from internal road segment, i				

WORKSHEET 3--AREA SOURCE EMISSIONS COMPUTATION
(see instructions following)

Project No.: Example 3 (con't)
Analyst: GVS
Site: Shopping Mall
Date: 6-6-78

Step	Symbol	Input/Units				
1	Brt	Base running time				
1.1		Base approach time(s)				
1.2		Base entrance time(s)				
1.3		Base movement-in time(s)				
1.4		Base stop, base start time(s)				
1.5		Base movement-out time(s)				
1.6		Base exit time(s)				
1.7		Base departure time(s)				
1.8		Total base running time(s)	270			
2	A	Area of parking lot (m^2)	2.34×10^5			
3	i	Entrance approach identification	N18			
4	Ve_i	Entrance demand volume (vph)	432			
5	Ce_i	Entrance approach capacities (vph)	1200			
6	i	Exit approach identification	N18			
7	Vx_i	Exit demand volume (vph)	304			
8	Cx_i	Exit approach capacities (vph)	1200			
9		Number of parking spaces occupied	1600			
10	F	Emissions	.63			
11	Pc	Capacity of parking lot (veh)	6000			
12	Rmi	Excess movement-in time(s)				
13	Fet	Facility emptying time(s)				
14		Excess running time				
14.1	Ve_i/Ce_i	Entering volume-to-capacity ratio	.36			
14.2	Vx_i/Cx_i	Exiting volume-to-capacity ratio	.25			
14.3	Re_i	Excess running time entering parking lot	1.7			
14.4	Rx_i	Excess running time exiting parking lot	1.0			
15	Te_i	Total entering running time (s/veh)	137			
16	Rmo	Excess running time moving out of parking stalls (s/veh)	0			
17	Tx_i	Total exiting running time (s/veh)	136			
18	Qa	Total emission rate from a parking lot ($g/m^2 - s$)				
19	Qa'	Area source emission rate without the emissions from internal road segment, i				

WORKSHEET 6--CO AREA SOURCE DISPERSION ANALYSIS
(see instructions following)

Project No.: Example 3

Analyst: G.J.S

Site: Shopping mall

Date: ______

Step	Symbol	Inputs/Units				
		Basic Inputs	Traffic Stream			
1		Source ID	AREA S-W		AREA N-E	
2	SC	Stability class	D		D	
3	U	Wind speed ($m\ s^{-1}$)	1.79		1.79	
4	σ_{z_o}	Initial dispersion (m)	5		5	
5	x_o	Virtual dispersion distance (m)	20		20	
6	x_u	Actual upwind distance (m)	60		340	
7	$r_u = x_u + x_o$	Effective upwind distance* (m)	80		360	
8	x_d	Actual downwind distance (m)	0		220	
9	$r_d = x_d + x_o$	Effective downwind distance* (m)	20		240	
10	Qa	Emission rate ($g\ m^{-2}\ s^{-1}$)	.000354			
		Dispersion Computation				
11	$(\chi U/Qa)_u$	Upwind normalized concentration	19000		26800	
12	$(\chi U/Qa)_s$	Downwind normalized concentration	-13600	-	-24400	-
13	$\chi U/Qa$	Normalized CO concentration*	5400		2400	
14	Qa	Emission rate ($g\ m^{-2}\ s^{-1}$)	x.00036	x	x.00036	x
15	χU		1.94		.86	
		Enter line 3	÷ 1.79	÷	÷ 1.79	÷
16	χ	CO concentration ($mg\ m^{-3}$)	1.08		.48	
17	χ	CO concentration (ppm)	.94		.42	

*Use Table 11 to determine $\chi u/Qs$ if $r > 500$ m and skip Steps 11 and 12.

V. EVALUATION TECHNIQUES FOR ESTIMATING CONCENTRATIONS DUE TO INDIRECT SOURCES

A. Introduction

Three basic source configurations generate vehicular emissions at indirect sources:

- Infinite line sources (i.e., free flow).
- Finite line sources (e.g., intersections).
- Area sources (e.g., parking lots).

The CO concentrations projected through application of these guidelines may arise from emissions contributions by any or all of the three source types. To evaluate the performance of the guidelines, data bases that provide CO, traffic, and meteorological data are used to derive and assess estimates of 1-hour CO concentrations for each source type. Four such data sets have been used for this purpose:

- Infinite line source--Bayshore Freeway, Santa Clara, California.
- Finite line source--Intersection of Route 83 and 22nd Street,
  Oakbrook, Illinois.
- Area sources--Liberty Tree Mall, Danvers, Massachusetts;
  Tacoma Mall, Tacoma, Washington

In the original application of these guidelines Supplement 5 to AP-42 and the 1974 Modal Model (Kunselman, 1974) were used in the evaluation. Since then, new mobile source emission factors for AP-42 (EPA, 1978) and revised Modal Model estimates have been added to the guidelines. Only limited correcting of this original evaluation, however, has been carried out in order to expedite this guideline. For infinite line sources the original evaluation using Supplement 5 is included here with discussion on the effects of the new emission factors. For finite line and area sources the new factors have been included.

B. Infinite Line Source Evaluation

1. Site Description

Traffic and meteorological data were obtained near the Bayshore freeway (U. S. Highway 101) by Dabberdt (1976). Figure 27 illustrates the location of the CO monitors as well as the configuration of the six-lane, at-grade freeway. The CO measurements from five samplers on each side of the road were used to evaluate the Guidelines for 18 one-hour periods; a total of 82 observations were considered.

2. Analysis

The Guidelines were used to estimate CO concentrations by considering separately the eastbound and westbound contributions and then summing the two. Figure 28 is a scatter diagram of observed and estimated concentrations; shown next to each data point is the corresponding value of the cross-roadway component of the wind speed. Note that the relationship between observations and estimates is generally within a factor of two or three. Much of the scatter occurs with wind speeds less than 1 m sec^{-1} which is to be expected with light, fluctuating winds. Other data with greater than 1 m sec^{-1} winds are generally close to the 1:1 line. These estimates, then, are very reasonable in light of state-of-the-art modeling techniques. Table 12 tabulates the case-by-case correspondence between observations and estimates. Out of 82 comparisons 46 estimates are within ± 2 ppm and 56 are within ± a factor of two. Many of the comparisons outside these limits can be related to the wind speeds below 1 m sec^{-1} as mentioned above.

To further assess a possible wind speed dependence (the distribution of wind values suggests a systematic variation), the relative

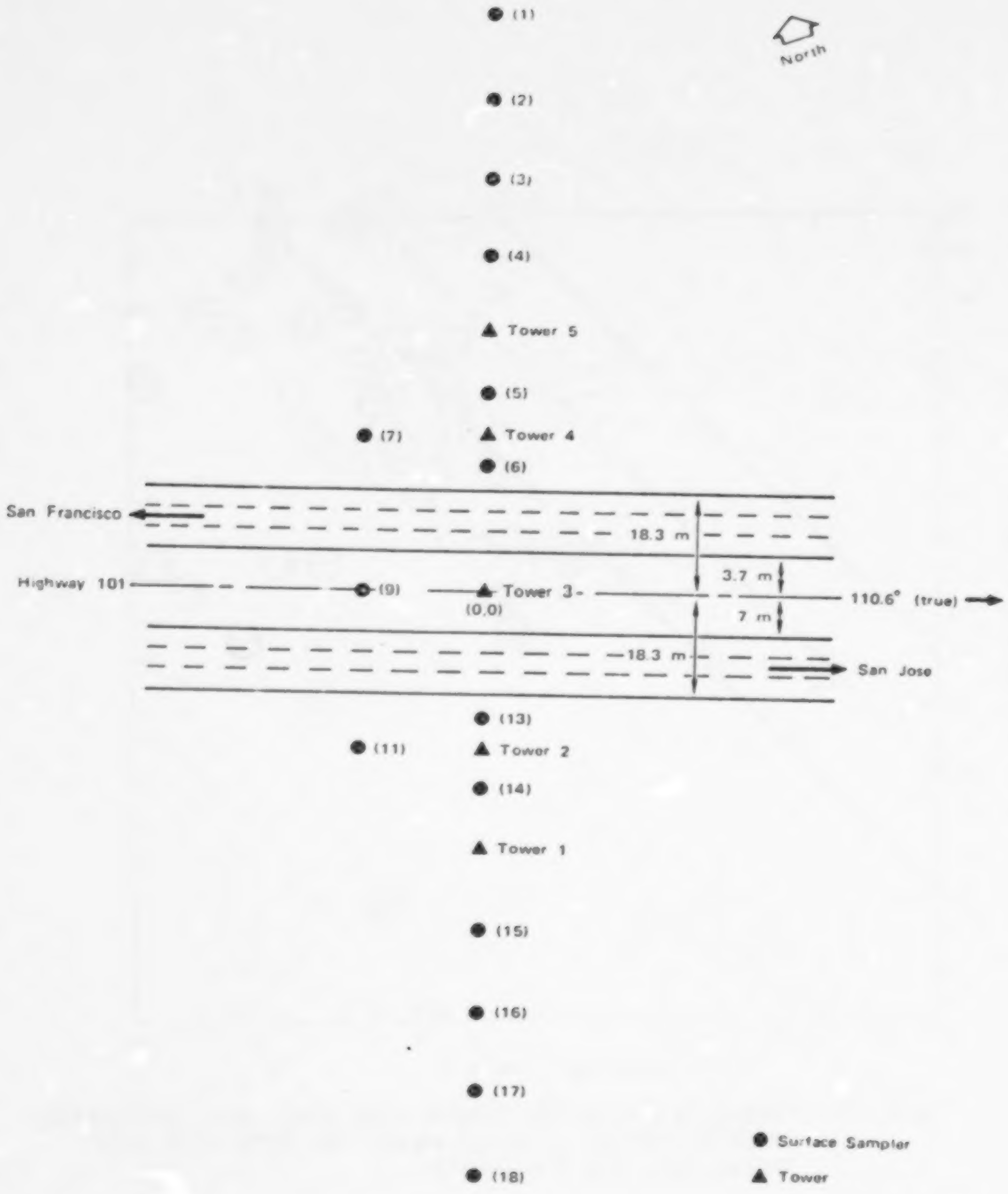
(1)
North
(2)
(3)
(4)
Tower 5
(5)
(7)
Tower 4
(6)
San Francisco
18.3 m
Highway 101
(9)
Tower 3
(0,0)
3.7 m
7 m
110.6° (true)
18.3 m
San Jose
(13)
(11)
Tower 2
(14)
Tower 1
(15)
(16)
(17)
(18)
● Surface Sampler
▲ Tower

Figure 27 . PLAN VIEW OF AIR QUALITY SAMPLER LOCATIONS FOR BAYSHORE FREEWAY STUDY

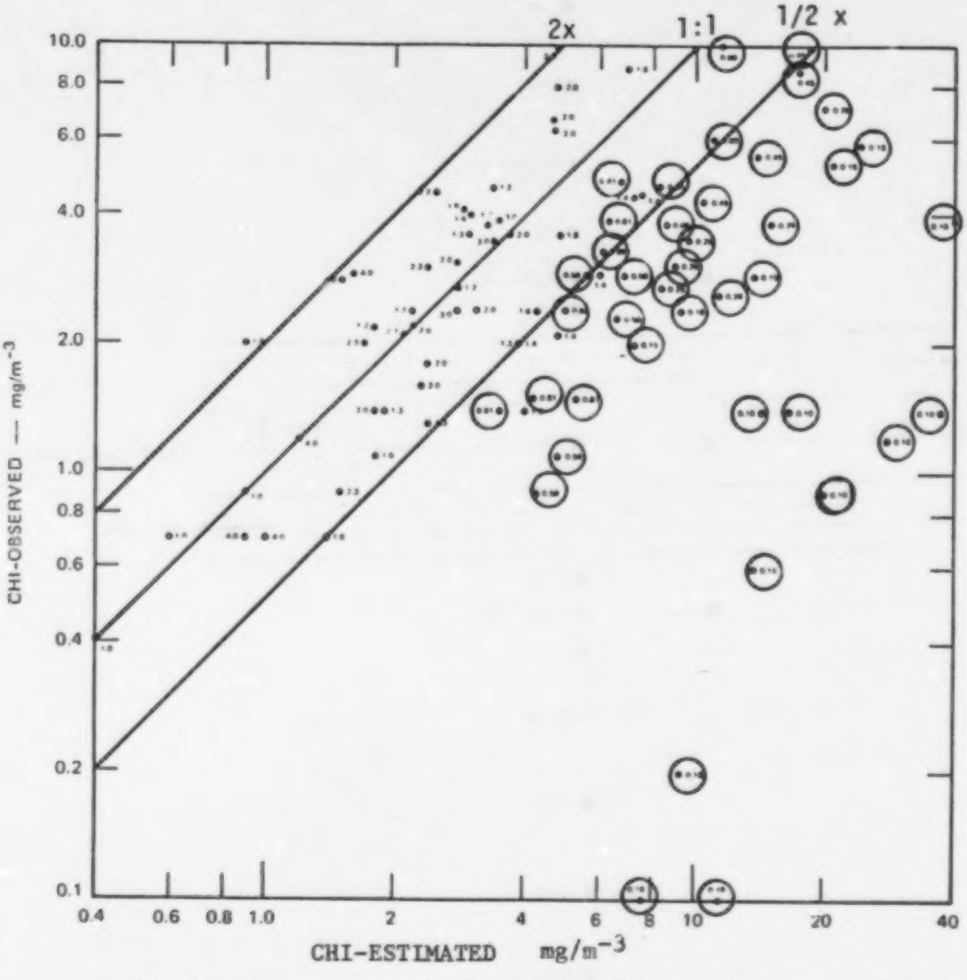

FIGURE 28. FREE-FLOW EVALUATION SHOWING WIND SPEED (m/s) DEPENDENCE: BAYSHORE FREEWAY (CIRCLED VALUES ARE THOSE WITH WIND SPEEDS LESS THAN 1 m sec^{-1})

Table 12
OBSERVED VERSUS ESTIMATED FREE-FLOW CO CONCENTRATIONS: BAYSHORE FREEWAY

Date	Hour	Receptor	CO CONCENTRATION (mg m^{-3})	
			Observations	Estimates
1-21-75	06	6	5.8	24.3
		141	5.2	20.9
		151	2.9	13.7
		3	2.4	9.3
		1	2.0	7.2
	07	6	9.8	18.5
		141	8.6	17.4
		151	5.5	14.0
		3	4.3	10.5
		1	4.7	8.3
1-24-75	06	6	4.8	6.7
		141	3.9	6.3
		151	1.5	5.3
		3	1.5	4.2
		1	1.4	3.5
	07	6	10.2	11.7
		141	6.0	11.1
		151	3.8	8.6
		3	3.3	6.1
		1	2.4	5.0
	08	6	7.1	20.0
		151	3.8	15.2
		3	2.6	11.4
		1	3.1	9.0
1-28-75	05	6	2.0	0.9
		141	0.9	0.9
		151	0.7	0.6
		3	0.4	0.4
	06	6	3.1	4.0
		141	2.0	3.8
		151	3.6	3.0
		3	1.3	2.4
		1	1.4	1.9
	07	6	7.9	4.9
		141	6.3	4.7
		151	3.6	3.7
		3	3.1	2.8
		1	2.2	2.2

Table 12 (continued)

Date	Hour	Receptor	CO CONCENTRATION ($mg\ m^{-3}$)	
			Observations	**Estimates**
1-28-75	08	6	4.5	2.5
		141	3.0	2.4
		151	2.1	2.1
		3	2.0	1.7
		1	0.9	1.5
1-30-75	15	13	4.0	3.0
		121	4.1	2.9
		16	1.1	1.8
		18	0.7	1.4
	16	13	9.8	4.8
		121	6.7	4.7
		16	2.4	3.1
		18	1.8	2.4
	17	13	11.0	7.2
		121	8.7	7.0
		16	3.6	4.9
		18	2.4	4.3
	18	13	4.6	3.4
		121	3.8	3.3
		111	2.7	2.8
		16	2.4	2.2
		18	2.2	1.8
	19	141	3.5	9.7
		151	2.7	8.4
2-5-75	15	6	2.9	1.6
		141	2.8	1.5
		151	1.2	1.2
		3	0.7	1.0
		1	0.7	0.9
	16	6	3.9	3.5
		141	3.5	3.4
		151	2.4	2.8
		3	1.6	2.3
		1	1.4	1.8
	17	6	4.5	7.5
		141	4.5	7.2
		151	2.9	6.0
		3	2.1	4.9
		1	2.0	3.9

Table 12 (continued)

Date	Hour	Receptor	CO CONCENTRATION ($mg\ m^{-3}$)	
			Observations	Estimates
2-5-75	18	6	2.9	6.9
		141	2.3	6.6
		151	2.9	5.6
		3	1.1	4.8
		1	0.9	4.3
		$\bar{X}$	3.45	5.75
		σ	2.37	4.99
		r		0.561
		$b^{(1)}$		1.919
		$m^{(1)}$		0.267
		rms		4.72
		N	82	82

difference between observed (χ_{obs}) and estimated (χ_{est}) values is plotted in Figure 29.

The relative difference ($\Delta\chi$) is normally given as:

$$\Delta\chi = \frac{\chi_{est} - \chi_{obs}}{\chi_{obs}} . \qquad (38)$$

However, when concentrations are low, even a difference as little as 1 ppm will show a large relative error (while the absolute differential may be insignificant). Differences of 1 ppm may result from sampling inaccuracies and thus may not always reflect real estimate errors. To minimize this low-concentration bias, the relative difference has been redefined, where

$$\Delta\chi = \frac{\chi_{est} - \chi_{obs} \pm 1}{\chi_{obs}} . \qquad (39)$$

The units are mg m^{-3}. The sign of the last term is taken as the opposite of the sign of the difference: $\chi_{est} - \chi_{obs}$. This convention acts as a filter to minimize the relative difference when the absolute difference is small. Figure 29 illustrates a possible nonlinear dependence of the relative concentration difference on the crossroad wind speed, especially if wind speeds less than 1 m sec^{-1} are considered.

More importantly the figure shows that at windspeeds above 1 m sec^{-1} the dependence of the relative concentration difference on the crossroad wind speed is nearly linear. This points up the importance of only considering windspeeds greater than 1 m sec^{-1} in these Guidelines. On the whole, for an infinite line source this validation demonstrates fairly good agreement between estimated and observed values for windspeeds greater than or equal to 1 m sec^{-1}.

3. New Emission Factors

Introducing the new emission factors (EPA 1978) and the revised Modal Model (1977) into the infinite line source evaluation will decrease the CO concentration estimates about 10%-15%. Hence, the results obtained above will change somewhat with the revised emissions estimates but are reasonable representations even with the required decrease.

C. Finite Line Source Evaluation

1. Site Description

Traffic and aerometric data for the evaluation of the finite line source methodology were obtained from a report by Patterson and

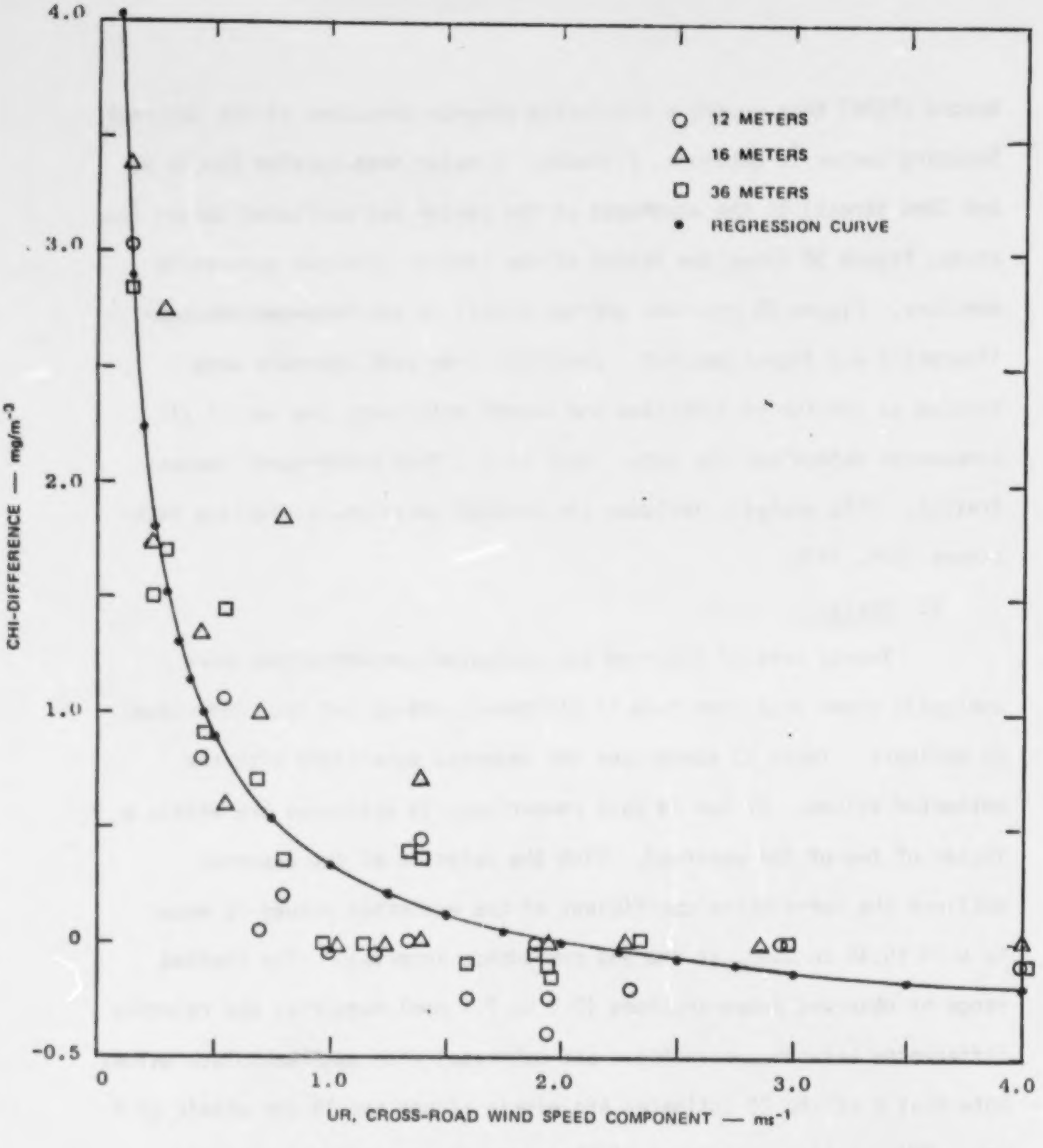

FIGURE 29. VARIATION OF DIFFERENCE BETWEEN OBSERVED AND ESTIMATED FREE-FLOW CONCENTRATIONS AS A FUNCTION OF CROSS-ROADWAY WIND SPEED

Record (1974) that covers a monitoring program conducted at the Oakbrook Shopping Center in Oakbrook, Illinois. A major intersection (Route 83 and 22nd Street) to the southwest of the Center was monitored during the study; Figure 30 shows the layout of the intersection and aerometric monitors. Figure 25 provides greater detail on the intersection configuration and signal phasing. Emissions from each approach were treated as the sum of free-flow and excess emissions; the sum of all components determined the total local (i.e., less background) concentration. This analysis includes the updated emissions estimating techniques (EPA, 1978).

2. Analysis

Twenty sets of observed and estimated concentrations were analyzed; these data came from 11 different periods and four individual CO monitors. Table 13 summarizes the observed data along with the estimated values. Of the 20 data comparisons 18 estimates are within a factor of two of the observed. With the deletion of two apparent outliers the correlation coefficient of the estimated values is equal to 0.73 (0.45 to 0.90, at the 95% confidence interval). The limited range of observed concentrations (2.3 to 7.1 ppm) magnifies the relative differences between observations and estimates with small absolute values. Note that 8 of the 20 estimates are within ±1 ppm and 18 are within ±2.5 ppm. This scatter may change if the range of concentrations is much larger, but further evaluation is needed to investigate this point.

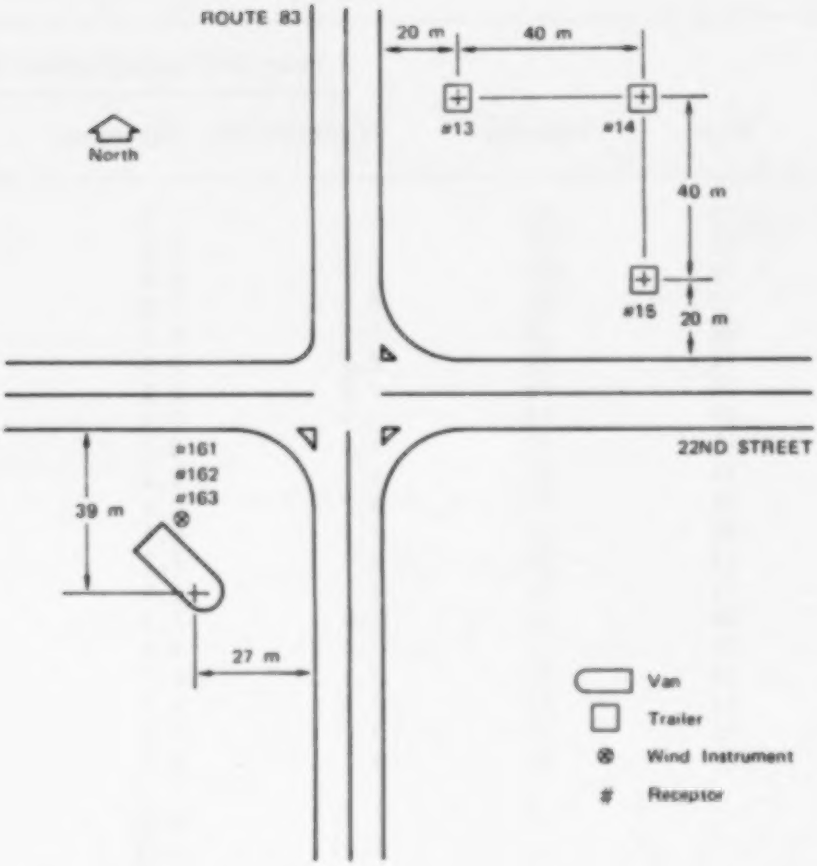
ROUTE 83
20 m
40 m
#13
#14
North
40 m
#15
20 m
22ND STREET
#161
#162
#163
39 m
27 m
Van
Trailer
Wind Instrument
# Receptor

FIGURE 30. CONTINUOUS MONITORING SITES AT THE ROUTE 83 – 22ND STREET INTERSECTION: OAKBROOK, ILLINOIS

Table 13

OBSERVED VERSUS ESTIMATED CO CONCENTRATIONS
OAKBROOK INTERSECTION

			1 Hour CO Concentration (ppm)	
Date	Hour	Receptor	Observations	Estimates
4-5-74	18	162	6.5	7.8
3-28-74	10	162	2.3	9.9
3-29-74	10	162	4.7	6.6
3-26-74	17	162	4.9	7.0
4-2-74	08	14	5.8	6.9
4-13-74	14	14	2.6	3.4
4-13-74	15	14	2.6	2.8
4-2-74	08	15	2.4	7.3
4-13-74	16	15	2.6	4.7
4-13-74	14	15	5.2	6.1
4-13-74	15	15	5.2	4.6
4-9-74	18	15	7.1	6.1
4-6-74	13	15	4.4	6.1
4-6-74	11	15	4.1	6.1
4-2-74	08	13	5.8	7.3
4-13-74	16	13	3.4	4.6
4-13-74	14	13	3.5	5.7
4-13-74	15	13	4.3	4.7
4-9-74	18	13	3.6	3.5
4-6-74	13	13	4.4	3.7
		$\bar{X}$	4.27	5.75
		σ	1.41	1.76
		r		.25
		N	20	20
		$\bar{X}$	4.48	5.43
		σ	1.31	1.48
		r		.73
		N	18[†]	18[†]

[†] Excluding Cases 2 and 8 as outliers.

D. Area Source Evaluation

1. Site Descriptions

Two regional shopping centers were the sites for traffic and aerometric studies that provided data for the evaluation of the area source methodology. Figure 31 shows the general layout of the Tacoma Mall in Tacoma, Washington, including the location of the CO monitors and anemometers. Of the 15 designated CO monitoring locations, only eight or less were operated simultaneously. Various upwind and downwind CO levels for the different periods monitored were compared. The upwind recordings showed large variability indicating that the empirical data may not always provide a reliable measure of background CO. In fact, an in-depth analysis by Mathematical Sciences Northwest (1974) relied instead on the results of a statistical analysis to provide an estimate of background CO concentrations. Even these estimates, however, were not obviously consistent with the measured downwind levels in all cases. As a result, background CO levels estimated subjectively from the available data introduced some uncertainty in the evaluation. Another result of the comparisons was that variations among downwind CO levels were often significantly large thus implying either a large inhomogeneity in area source emissions over the fetch upwind of the monitors, or the possible influence of significant intraparking lot line sources.

The second area source data base was obtained at the Liberty Tree Mall near Boston, Massachusetts. Figure 32 shows the principal

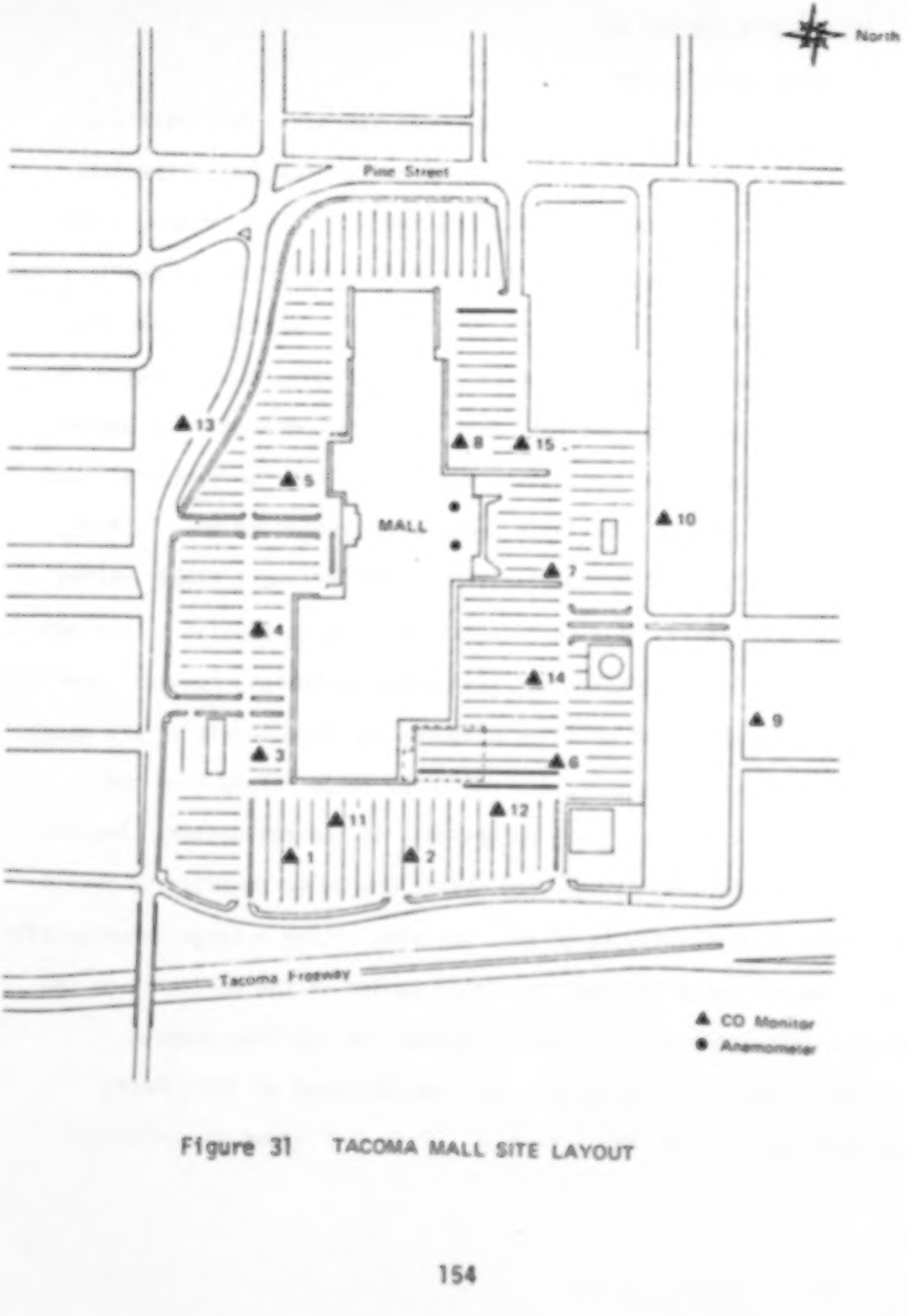
North
Pine Street
MALL
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Tacoma Freeway
▲ CO Monitor
● Anemometer

Figure 31 TACOMA MALL SITE LAYOUT

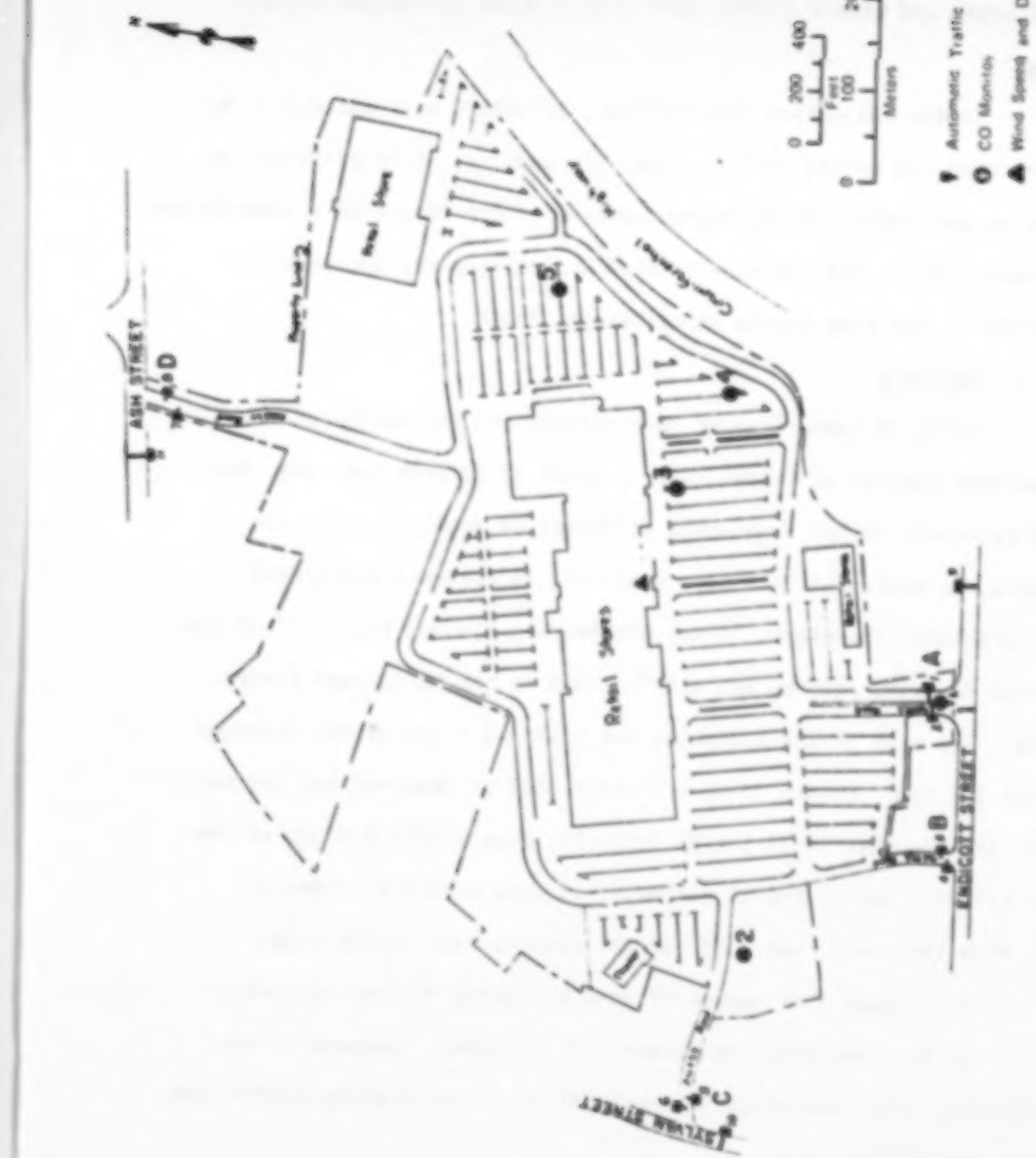
ASH STREET
D
ENDICOTT STREET
A
B
SYLVAN STREET
C
Retail Stores
Retail Store
2
3
4
5
0 200 400
Feet
0 100 200
Meters
Automatic Traffic Counter
CO Monitor
Wind Speed and Direction

Figure 32 . LIBERTY TREE MALL SITE LAYOUT features of the Center along with the location of the traffic, wind, and CO monitors. The background and downwind concentrations reported by Patterson and Record (1974) were used in this evaluation without change.

In the evaluation that follows, emissions were assumed to be uniform over the entire parking area. In summary, it is difficult to obtain concentrations truly representative of the impact of a distributed emissions source, and the experimental uncertainty may aoproach the magnitude of the area source contribution itself.

2. Analysis

Table 14 summarizes CO observations and the estimates for 14 one-hour periods at Tacoma Mall; a total of 23 data sets are shown, which represent values from seven different CO monitors. For six of the sets, no empirical data were available to estimate background concentrations; therefore, it was arbitrarily assumed that half of the measured downwind reading was attributable to the background contribution. The mean of the estimates was within 0.1 ppm of the observed mean of 3.2 ppm. Figure 33 is a scatter plot of observed and estimated values showing that 14 of the 23 estimates were within 2.0 ppm of the observations. Out of the 23 estimates 15 were within a factor of two. With the great number of generalizations made in the area modeling techniques this number of good estimates is very reasonable even though the remaining ten values are scattered. Because of the experimental (and theoretical) uncertainties, a correlation coefficient was not computed.

Table 14

OBSERVED VERSUS ESTIMATED AREA SOURCE CO CONCENTRATIONS: TACOMA MALL

Date	Hour	Receptor	CO Concentration (ppm)[*]	
			Observations	Estimates
12-21-73	10	4	7.0	0.7
	10	8	1.0	0.8
	11	3	3.5	1.6
	11	8	2.0	2.3
	11	7	1.0	1.7
	18	8	2.5	4.2
	18	7	5.0	2.8
	19	8	6.0	3.1
	19	7	2.0	2.6
12-26-73	13	8	5.0	8.0
	13	7	2.0	6.1
	14	8	6.0	4.0
	14	7	0.0	3.0
	15	8	6.5	3.1
	15	7	3.5	2.3
12-21-73	12	4	8.0	3.4
	12	5	2.0	3.0
	13	5	1.8[+]	3.7
	14	5	3.0[+]	2.1
2-6-74	12	15	1.0[+]	1.6
	14	14	2.0[+]	1.2
	16	14	1.0[+]	2.9
	17	14	2.0[+]	7.9
		$\overline{x}$	3.21	3.13
		rms		2.78

[+] No reliable background; local area source contribution approximated as one-half total observed concentration.

[*] A constant 60°F (15°C) temperature was assumed because no temperature data was available.

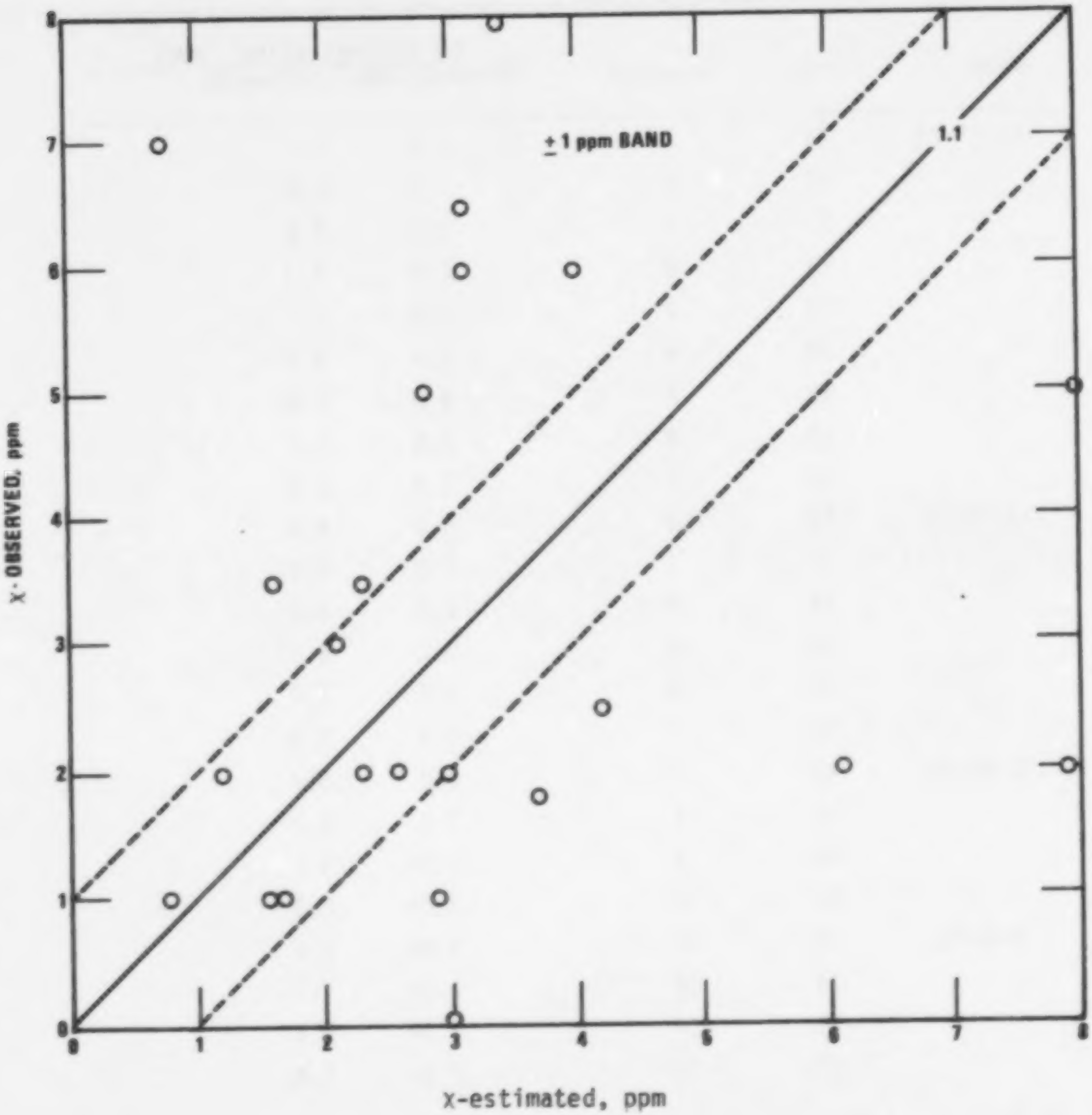

Figure 33. Comparison of observed and estimated concentrations at the Tacoma Mall.

Results of the area source analysis for Liberty Tree Mall are summarized in Table 15; Figure 34 is a scatter diagram of observed and estimated values for the 10 one-hour cases evaluated. Six of the ten cases agree within 3 ppm. Also six of the estimated values are within a factor of two of the observed values.

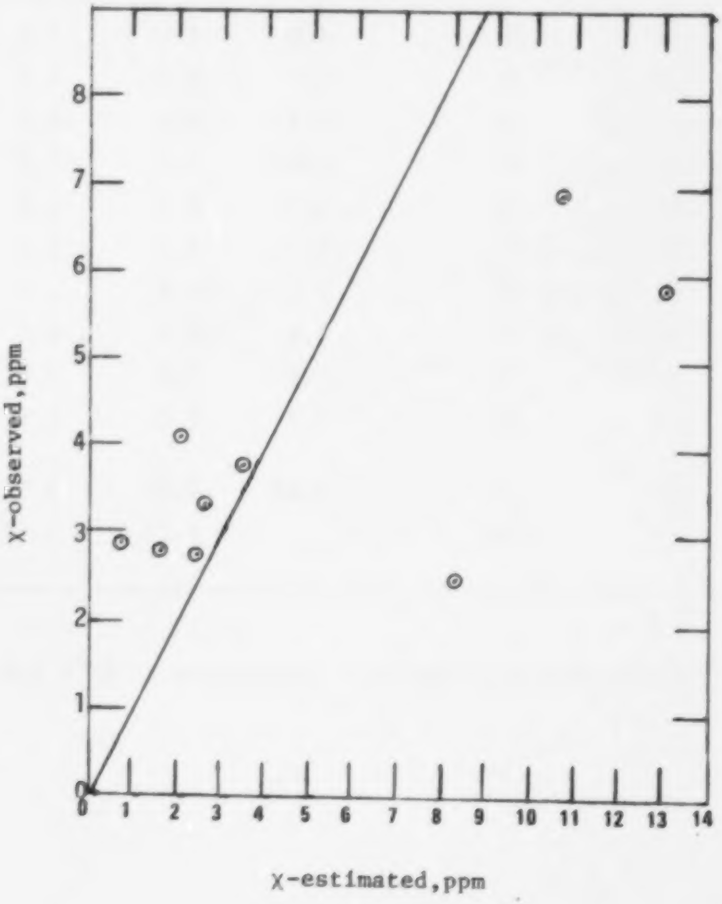

Figure 34. Comparison of observed and estimated concentrations at Liberty Tree Mall.

Table 15

OBSERVED VERSUS ESTIMATED AREA SOURCE CO CONCENTRATIONS: LIBERTY TREE MALL

			CO Concentration (ppm)		
Date	Hour	Receptor	Observations	Estimates	Adjusted[†]
12-11-73	19	4	4.2	2.1	1.7
12-12-73	19	4	6.9	10.5	5.1
12-13-73	19	3	3.3	2.6	2.2
12-14-73	19	4	2.8	1.7	1.7
12-15-73	14	4	2.4	8.3	8.9
12-18-73	19	4	3.7	3.4	2.0
12-19-73	19	4	3.5	23.2	8.1
12-20-73	19	4	5.8	13.1	6.8
12-21-73	19	3	2.9	0.8	0.9
12-22-73	14	4	2.7	2.4	1.9
		$\bar{x}$	3.82	6.80	3.93
		rms		6.71	3.43

[†] Estimated value adjusted to a constant temperature of 60°F (15.5°C).

Concentration estimates differ between Liberty Tree Mall and Tacoma Mall due to: the volume demand is about 20% greater at Liberty Tree, the number of parking spaces is about 16% less at Liberty Tree, and the cold-start/temperature factors are applied inconsistently. At Tacoma, no temperature data were readily available, and so a constant temperature (60°F (15.5°C)) was assumed. Temperatures at Liberty Tree Mall were considerably less, and the corresponding adjustment factors were substantially greater. To put the estimates on a consistent basis, an 'adjusted' estimate based on a constant 60°F (15.5°C) temperature was applied to the Liberty Tree values with the hour-specific cold-start ratios. These revised values are tabulated in Table 15. This comparison is presented only to show that under consistent application, the estimated concentrations are of consistent magnitude. It also shows, however, that the temperature correction may in part be responsible for some of the overestimation. In view of other problems in interpreting data, further investigation is needed.

E. Summary

The indirect source review methodology has been evaluated on the basis of experimental data representative of infinite line sources, finite line sources, and area sources. Comparison of observations and estimates for the infinite line source gave relative differences generally within a factor of two. Many of the points greater than this factor of two were found to occur at wind speeds less than 1 m sec^{-1}. In view of the state-of-the-art of dispersion modeling these results seem very reasonable especially at wind speeds greater than 1 m sec^{-1}. The inaccuracies associated with lower wind speeds require further investigation using empirical data from several sites. If such investigations confirm a wind speed dependence, an adjustment procedure might be justified.

Observations at the intersection of two major arterials provided the data base for the evaluation of the finite line source assessment methodology. The observed CO concentrations attributable to emissions from the intersection ranged from 2 to 7 ppm. Correlation with estimates was 0.73; while the rms difference equaled 1.3 ppm for the estimates values. On an absolute basis, the estimates only underestimated by an average of 0.5 ppm. Another measure of the performance of the methodology is the large number (18 of 20) concentration estimates that are within ± ppm of the observed.

Evaluation of the area source methodology provided a reasonable demonstration of the Guideline techniques. Of the 33 estimates made for two shopping center complexes, 20 were within a factor of two of the observed values. Overestimates may be in part due to the cold-start/temperature corrections applied in the technique, but further investigation into the sensitivity of all factors is needed.

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Appendix A

SITE-SPECIFIC TRAFFIC PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS[1]

This appendix provides guidance for persons estimating and computing the various traffic parameters used in the assessment methodology of Chapter IV; seven types of indirect emissions sources are discussed:

- Highways
- Airports
- Shopping centers
- Sports complexes
- Municipal parking lots
- Amusement parks
- Recreational areas.

For the most part, the information contained in this appendix has been extracted from Appendices A through C of the EPA "Guidelines for Air Quality Maintenance Planning and Analysis, Vol. 9: Evaluating Indirect Sources" (1975).[2] That source and the references cited at the end of this appendix should be consulted for additional site-specific guidance.

1. Highways

This section provides guidance for air pollution control agencies determining input traffic parameters required for estimating CO

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[1] Several letter symbols in this appendix are used differently than those in the main text. All symbols are defined, however, to avoid confusion.

[2] References appear at the end of the appendix.

(carbon monoxide) emissions resulting from the use of a new or improved highway. The values of key design and operating variables depend heavily on local conditions. Therefore, these parameters must be assessed on a case-by-case basis by personnel familiar with traffic engineering concepts. Once values for the key variables are estimated, they can be related to emissions, using the methodology described in this section.

a. Input Parameters

Table A-1 presents key variables which, are provided by the developer or appropriate highway department. Road segments can be conveniently defined as the portions of roads between adjacent exits/entrances. In analyzing the impact of a highway on air quality, it is desirable to divide the highway into segments, because the traffic volume, demand and the nature of the traffic (e.g., number of trucks) are likely to change from segment to segment. Such changes, in addition to imposing varying demands on the capacities needed to avoid congestion (and the resulting higher emissions), may actually affect the capacity itself. For example, if one segment has more truck traffic than another, its overall capacity will be less, all other factors being equal. The length of the road segments can be determined directly from the plans provided by the developer. Each direction would be considered separately in dividing the road into segments.

Among the variables in Table A-1, traffic capacity per lane per road segment is a key design parameter because it provides a means for determining when traffic volume demands will result in congestion, thereby causing high emissions. Design speed is primarily of importance in determining speeds (and therefore emission factors) under various volume demand-to-capacity ratios. Vehicle speed and volume demand-to-capacity ratios determine the level of service occurring on a road. Right-of-way is of interest in selecting sensitive receptor sites.

Table A-1 HIGHWAY PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS

Variable	Remarks
Traffic capacity per lane per road segment (including merge lanes)	A road segment is ordinarily that portion of a highway between adjacent entrance/exits. Capacity is consistent with level of service E.
Design speed	This variable ordinarily is the posted speed limit.
Right of way and median width	
Number of lanes per road segment	
Access road capacity per lane	Consistent with level of service E.
Number of lanes per access road (for key access roads) and number of merge lanes	A key access road is ordinarily one in which traffic volume demand approaches access road capacity.
Annual average daily traffic (AADT) for each segment	Even more desirable would be the AADT per lane per segment if the demand among lanes is expected to differ substantially.
Seasonal, weekly and diurnal use patterns	
Estimated peak 1-hour and 8-hour traffic volume demand per road segment	
Vehicle mixes utilizing segments	For example, age mixes and proportions of light, medium, and heavy-duty vehicles.
Plans or blueprints of the highway, its route and access roads	Needed to identify road segments for each direction
Average highway speed	Weighted average of design speeds within a highway segment.
Green time to signal cycle ratio	Used in estimating capacity at intersections.
Cycle length	The sum of time a traffic signal spends in each phase of the signal cycle. Useful in estimating queue lengths upstream from intersections.
Location of the road segment evaluated	Central business district, fringe areas, outlying business district, residential, rural areas.

Ordinarily, such a site would not occur within the right-of-way. It is important to know the number of lanes per road segment in applying a line source diffusion model to estimate CO concentrations in the vicinity of the highway. In addition, the number of lanes per segment is one of several determinants of the segment's capacity and is needed to estimate capacity if estimates of this parameter are not directly available. The number of lanes and capacities for access roads are important parameters because the peak impact of a major highway may occur at sensitive receptors in the vicinity of on or off ramps. The three operational parameters [annual average daily traffic (AADT), use rate patterns, and peak demands] are important, acting in two essential capacities:

- As estimates of the sheer numbers of vehicles using road segments.
- As indicators (in combination with capacity) of whether congestion is likely.

The vehicle mix utilizing each segment is a determinant of the appropriate emission factor per segment as well as the segment's capacity.

b. Traffic Volume Demand

Projected traffic volume demand (V_i) should reflect diverted and induced traffic as well as possible. Diverted traffic results from trips that are diverted from one or more other roads to the new or modified highway. For example, the presence of the highway might make it much more convenient to utilize shopping center A than B that is located in a different area. The result is an increase in traffic along the new route caused by people formerly using different routes to arrive at shopping center B. Induced traffic results from attractions that build up along the route of the highway and generate trips to them. The amounts of diverted and induced traffic depend entirely on local conditions, so only qualitative suggestions for their determination can be provided here. The economic-demographic determinants of traffic demand enumerated in Section 5.0 in "Guidelines for the Review of the Impact of Indirect Sources on Ambient Air Quality" (EPA, 1973) should be checked for relevance. Land-use and other maps plotting census information may provide additional insight in estimating diverted and induced demand.

The total traffic volume demand of road segment i can be estimated for 1- and 2-hour periods by beginning with $(AADT)_i$ and multiplying by peak seasonal adjustment factors. The peak seasonal average daily traffic rate would then be multiplied by the peak weekly 1-hour or 8-hour adjustment factor:

$$V_i = (AADT)_i \begin{pmatrix}\text{Seasonal adjustment}\\ \text{demand factor}\end{pmatrix} \begin{pmatrix}\text{Fraction that peak 1-hr demand is}\\ \text{of peak seasonal daily demand}\end{pmatrix} \tag{A1}$$

or

$$V_i = \frac{(AADT)_i \begin{pmatrix}\text{Seasonal adjustment}\\ \text{demand factor}\end{pmatrix} \begin{pmatrix}\text{Fraction that peak 8-hr demand}\\ \text{of peak seasonal daily demand}\end{pmatrix}}{8}, \tag{A2}$$

where V_i = traffic volume demand for road segment i, vehicles per hour (vph).

If the information on diurnal and weekly variations in traffic patterns is insufficient for using Eqs. (A1) or (A2), information in a report prepared for EPA (Thayer, 1973) can be used to derive Eqs. (A3) and (A4):

$$V_i = (0.094) \begin{pmatrix}\text{Seasonal adjustment}\\ \text{demand factor}\end{pmatrix} (AADT)_i \quad , \tag{A3}$$

$$V_i = (0.50) \frac{\begin{pmatrix}\text{Seasonal adjustment}\\ \text{demand factor}\end{pmatrix} (AADT)_i}{8} \quad . \tag{A4}$$

If seasonal changes are not known, it is suggested that the factors presented in Table A-2 be used in Eqs. (A3) or (A4).

Table A-2

SEASONAL ADJUSTMENT DEMAND FACTOR
FOR ESTIMATING PEAK ADT FROM ANNUAL ADT

Average Annual Daily Traffic (vehicles)	Peak Period, Urban Location		Peak Period, Rural Location	
	1-Hour	8-Hours	1-Hour	8-Hours
< 20,000	1.70	1.40	1.70	1.68
20,000 - 50,000	1.38	1.30	*	1.68
> 50,000	1.06	1.20	*	1.40

If the V_i calculated with Eqs. (A1) and (A2) did not take mass transit into account, the volume demand for 1- and 8-hour peak periods would have to be adjusted, using Eqs. (A5) and (A6):

$$V_i = \text{Eq. (A1)} - \frac{(T)\ (P)}{(Avo)} + B_i \qquad \text{(A5)}$$

$$V_i = \text{Eq. (A2)} - \frac{(T)\ (P)}{8\ (Avo)} + \frac{B_i}{8} \qquad \text{(A6)}$$

where

P = fraction of passengers normally using a private automobile where mass transit is not available,

Avo = average number of passengers per automobile

B_i = number of mass transit buses using road segment i during the selected 1- or 8-hour period,

T = number of passengers using buses during the selected 1- or 8-hour period.

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[*] Assume each lane accomodates 2000 vehicles per hour and traffic volume demand = (number of lanes) (2000).

V_i would then be apportioned among the lanes in segment i in accordance with such locally determined considerations as presence of bus lanes and volume demand at nearby exits and entrances.

Figure A-1 is a schematic drawing of one segment of one direction of an eight-lane highway. V represents volume demand (vph). The first numerical subscript refers to the road segment, while the second numerical subscript refers to the lane within the segment. The first lane is defined as the outside lane. The lower case letters are interpreted as follows: x stands for exiting traffic; e stands for entering traffic, and t stands for through traffic. Lf represents the length of a given road segment. For any road segment, the traffic volume demand is given by Eq. (A7):

$$Vt_i = Vt_{i-1} + Ve_i - Vx_i \ . \tag{A7}$$

Thus for road segment 1 in Figure A-1,

$$Vt_1 = Vt_o + Ve_1 - Vx_1 \ . \tag{A8}$$

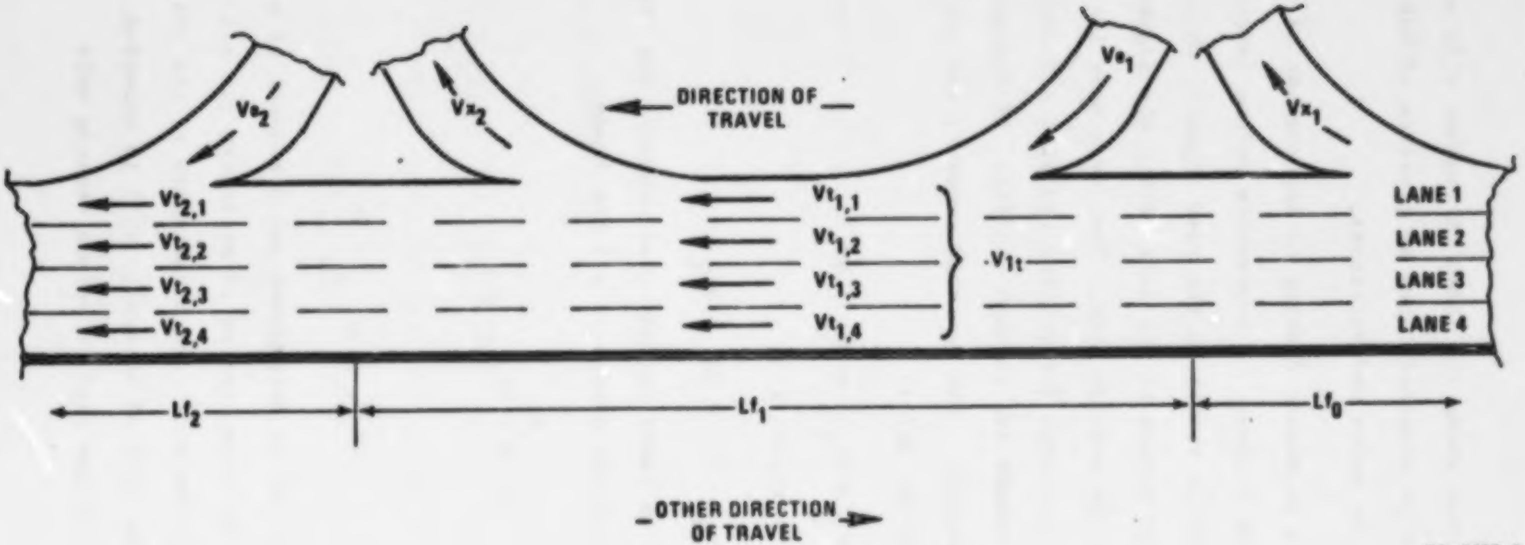
Also, if m is the number of lanes for any road segment, the traffic volume demand is the sum of the demand in all the lanes:

$$Vt_i = \sum_{j=1}^{m} Vt_{i,j} \ , \tag{A9}$$

or, as applied to Figure A-1,

$$Vt_1 = Vt_{1,1} + Vt_{1,2} + Vt_{1,3} + Vt_{1,4} \tag{A10}$$

The traffic volume demand can be determined for a segment if a direct estimate is provided by the developer or if estimates for all preceding exit and entrance demands are known and an estimate of the traffic volume demand entering the area of interest (Vt_o) is provided. It would be most convenient if the traffic volume demands were


Ve_2
Vx_2
DIRECTION OF TRAVEL
Ve_1
Vx_1
$Vt_{2,1}$
$Vt_{2,2}$
$Vt_{2,3}$
$Vt_{2,4}$
$Vt_{1,1}$
$Vt_{1,2}$
$Vt_{1,3}$
$Vt_{1,4}$
V_{1t}
LANE 1
LANE 2
LANE 3
LANE 4
Lf_2
Lf_1
Lf_0
OTHER DIRECTION OF TRAVEL

SA-4429-1

FIGURE A-1 SCHEMATIC DIAGRAM FOR ANALYSIS OF A MAJOR HIGHWAY approximately the same for each lane. Unfortunately, this frequently is not the case. Given the information that is usually available, it may only be possible to identify traffic flow by direction. If information is available on the amount of traffic entering and exiting (or turning on arterial roads), it may be used as a basis for more refined apportionments in each direction. In Figure A-1, the following definitions and relationships apply with respect to capacity. $Ct_{i,j}$ is the capacity of lane j in segment i:

$$Ct_i = \sum_{j=1}^{m} Ct_{i,j} \qquad \text{(A11)}$$

Capacity is determined by a number of factors, many of which can only be determined by examining actual plans, including information on such considerations as volume demands and capacities on succeeding road segments, demands and capacities on access roads, and presence of turning or exit/entrance lanes. There are also several determinants whose effects can be gauged on a more general basis. These include such operating and design parameters as lane width, lateral clearance, percentage of trucks utilizing the highway, design speed, and grade.

2. Airports

This section provides guidance for estimating ground traffic at airports that primarily handle operations of the commercial carrier type. It has been estimated (Norco, et al., January 1973) that within the next 10 years, 112 new airports of this nature will be built. Thus the development of major new airports to handle commercial carriers is a relatively rare event, and it is likely that a detailed environmental impact statement will be required for each such development.

a. Input Parameters

In a report prepared for EPA (Thayer, August 1973), possessing one of three sets of airport utilization parameters is considered essential for estimating emissions from ground traffic associated with airport activities. The preferable set is trip data. If these data are not available, then data concerning airport population or aircraft operations would have to serve as the starting point for deriving trip data and ultimately emission data. In any case, certain airport design information should be required from developers. This design information includes a schematic layout of the airport, indicating approximate dimensions, the number of public parking spaces available, the number of parking lot gates, gate capacities, terminal curb frontage, and frontage road capacity.

Table A-3 summarizes parameters that should be obtained. The parameters in group I-D as well as a complete set of group I-A, I-B, or I-C parameters are essential. It is desirable to have the information in group II, Useful Parameters, for an individual planned airport. However, if this information is not available, estimates can be made on the basis of data gathered for EPA for 13 major airports. If the planned airport does not cater primarily to commercial carrier operations, certain parameters in group II are essential, since the approximations in this section are based on observations at airports servicing primarily commercial carrier operations.

b. Traffic Volume Demand

Three basic approaches can be taken to estimate ground traffic demand generated by the airport's presence.

Trip Data--First, and most preferable, one can work directly with expected trip generation rates provided by the developer. Ideally, the trip generation estimates correspond to the periods of interest specified by applicable air quality monitoring data. On the federal level, for carbon monoxide this would mean:

Table A-3

COMMERCIAL AIRPORT PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS

Parameters	Remarks
I Essential Parameters	
A. Trip Data	
Average daily trip generation rate	A trip is a one-way trip to or from the airport. Thus a round trip is two trips.
B. Passenger/Employee/Visitor Parameters	
Average daily airport population	Includes passengers, visitors, and employees.
Estimated fraction of passengers on through flights or transferring planes	This parameter can vary markedly depending on airport location.
Fraction of passengers accomodated by mass transit	
Number of buses arriving at airport	
C. Aircraft Operations	
Annual number of LTO cycles	One LTO cycle is one landing plus one takeoff.
Estimated passenger seats per LTO cycle	This parameter depends on the mix of commercial aircraft classes.
Number of airport employees	This parameter is needed because no clear-cut relationship between number of employees and aircraft operations is discernible.

Table A-3 (continued)

Parameters	Remarks
I Essential Parameters (cont.)	
D. Design Parameters	
Number of public parking spaces	
Number of gates	
Gate capacities	
Gate traffic control characteristics	For example, green time to signal cycle ratios and signal cycle lengths.
Number of employee parking spaces	
Terminal curb frontage	
Terminal frontage road capacity	
Plans and/or blueprints	To provide a schematic picture of access roads, traffic lanes, and the dimensions of the complex.
II Useful Parameters	
Peak daily airport population	
Peak and average daily passenger population	
Peak and average daily visitor population	
Vehicles/airport population	
Peak and average daily employee population	
Peak and average daily vehicle population	
Percentage peak 1-hour and 8-hour vehicle trips of peak daily trips	

Table A-3 (continued)

Parameters	
II Useful Parameters (cont.)	
Percentage peak 1-hour and 8-hour employee trips of peak daily trips	
Percentage peak 1-hour and 8-hour passenger and visitor trips of peak daily trips	
Percentage vehicle trips, 6-9 AM, of peak daily vehicle trips	
Peak numbers of daily, hourly and 8-hour LTOs	
Number of LTOs, 6-9 AM on peak day	
Typical percentage of aircraft seating capacity filled	
Vehicles/LTO	
Peak daily trip rate	
Peak hourly trip rate	
Peak 8-hour trip rate	
Peak trip rate, 6-9 AM	
Peak trip rate 6 PM - 6 AM	
Yearly trip rate	
Base running time at airport	Typical running time by a vehicle with no congestion.
Average speed on airport access roads	Outside of the airport but within 3 miles of its boundary.

- The peak hourly trip generation rate
- The peak 8-hour trip generation rate.

Estimates of this sort may be difficult to obtain, however. Only annual average daily trip generation estimates may be available. In this case, data compiled for EPA (Thayer, August, 1973) for a sample of airports can be used to infer trip generation rates for appropriate periods. Equations (A12) to (A17) provide means for estimating trip generation rates for various periods of time; they were derived using information from Thayer (August,,1973) about the relative numbers of passengers, visitors, and employees on peak and average days and assume that employees' and visitors' trips are two way.

$$\text{PDT, vph} = 0.054 \text{ ADT} , \tag{A12}$$

$$\text{Peak hourly trip rate, vph} = 0.079 \text{ PDT} = 0.112 \text{ ADT} , \tag{A13}$$

$$\text{Peak 8-hour trip rate, vph} = 0.061 \text{ PDT} = 0.083 \text{ ADT} , \tag{A14}$$

$$\text{Peak trip rate, 6 - 9 AM, vph} = 0.040 \text{ PDT} = 0.050 \text{ ADT} , \tag{A15}$$

$$\text{Peak trip rate, 6 PM - 6 AM, vph} = 0.029 \text{ PDT} = 0.036 \text{ ADT} \tag{A16}$$

$$\text{Yearly trip rate, vph} = 0.043 \text{ ADT} \tag{A17}$$

where

PDT = Peak daily trip rate, trips per day,
ADT = Annual average daily trip rate, trips per day.

Airport Population Data--In some instances, a developer may only have airport population estimates rather than direct estimates of trip generation rates. Airport populations are divided into three distinct categories: passengers, visitors, and employees. Obviously, if there is specific information available about peak and typical numbers for each category, it should be used. Otherwise, the information in Table A-4, which has been compiled by SRI from a sample of 13 major airports, will have to suffice as a basis for estimating peak and typical numbers of passengers, visitors, and employees present.

Table A-4 ESTIMATES OF PEAK AND TYPICAL NUMBERS OF PASSENGERS, VISITORS, AND EMPLOYEES IN AIRPORT POPULATIONS

	Fraction of Typical Daily Airport Population		
	Passengers	Visitors	Employees
Typical day	0.45	0.33	0.22
Peak day	0.60	0.52	0.23

Estimation of traffic volume from population data proceeds as follows. First, the number of passengers, visitors, and employees on peak and typical days are computed by multiplying a typical day's airport population by the appropriate factors in Table A-4. The second step is to estimate the number of vehicles used by passengers, visitors, and employees. For passengers, this number depends greatly on the fraction of passengers simply transferring planes or on through flights. These passengers do not generate any ground traffic at all. This fraction varies considerably, depending on the location of the airport. It is essential that the developer provide an estimate of the fraction of through and transferring passengers in order for the control agency to successfully relate airport population to vehicle population. Once the fraction of transferring and through passengers is known, Eq. (A18) can be used to estimate the number of vehicles used by passengers and and visitors:

$$(\text{Veh})_{pv} = 0.75\ (\text{total passengers per day})\ (1-F_t) \quad , \qquad (\text{A18})$$

where

$(Veh)_{pv}$ = the number of vehicles used per day by passengers and visitors,

F_t = the fraction of through and transferring passengers.

The parameter $(Veh)_{pv}$ can be adjusted to reflect increased use of mass transit by applying Eq. (A19):

$$(Veh)_{pv} = 0.75\ [(\text{total passengers})\ (1-b)]\ (1-F_t) + B\ , \qquad (A19)$$

where

b = fraction of passengers using mass transit,

B = number of buses running during period of interest.

The number of vehicles used by employees $(Veh)_E$ can be estimated using Eq. (A20):

$$(Veh)_E = 0.82\ (\text{number of employees present per day}) \qquad (A20)$$

Next, ADT can be estimated from the information present in Table A-4, and Eq. (A18) to (A20). In estimating ADT, it is necessary to apportion $(Veh)_{pv}$ among passengers who park their own vehicles and those who are dropped off or picked up by visitors. From Table A-4, the total number of passengers originating or terminating their flights at the airport on an average day is:

$$Po = 0.45\ (\text{total average population})\ (1-F_t)\ , \qquad (A21)$$

and the total number of visitors is:

$$Av = 0.33\ (\text{total average population})\ . \qquad (A22)$$

Thus, of the total number of vehicles driven by passengers and visitors, the fraction of vehicles driven by visitors (F_v) is:

$$F_v = \frac{0.33}{0.33 + 0.45\ (1-F_t)} \qquad (A23)$$

and the fraction driven by passengers (F_p) is:

$$F_p = \frac{0.45\ (1-F_t)}{0.33 + 0.45\ (1-F_t)} \quad . \qquad \text{(A24)}$$

If the assumption is made that vehicles driven by visitors and employees make two trips per day (one coming, one going) and that vehicles driven by passengers make one trip per day, then the average daily traffic is given by Eq. (A25):

$$ADT = 2F_v(Veh)_{pv} + F_p(Veh)_{pv} + 2(Veh)_E \qquad \text{(A25)}$$

The results of Eq. (A25) can now be used in Eqs. (A12) to (A17) to estimate traffic volumes for appropriate periods of interest.

Aircraft Operation Data--The third and least preferable approach to estimating traffic volume is using aircraft operation data to estimate ground traffic demand generated by passengers and visitors and using employee data to estimate that generated by employees. As with the other two approaches, if the developer is able to provide reliable estimates of the various Useful Parameters of Table A-3, they should be used directly rather than derived from information based on EPA's limited sample of airports. Starting with the annual number of LTO cycles (One LTO cycle is one landing plus one takeoff operation), the average daily number of LTO cycles is simply:

$$\text{Daily LTO cycles} = \frac{\text{Annual LTO cycles}}{365} \qquad \text{(A26)}$$

The other required parameter, passenger seats per LTO cycle, can be estimated by anticipating the mix of aircraft likely to be using the airport. Table A-5 presents different classes of aircraft and the mean of mixes observed at four large airports (Thayer, August 1973).

Table A-5

AIRCRAFT CLASSES AND THEIR OBSERVED UTILIZATION AT FOUR LARGE AIRPORTS

Aircraft Class	Seating Capacity	Percentage of Aircraft in Total LTO Cycles (1969)
SST (supersonic transport)	136	0%
Jumbo jet	490	0
Long-range jet	129	38
Medium-range jet	116	49
Turbo-prop	61	13
Business jet	10	0
Piston engine utility	1	0

Source: Thayer (August 1973).

In the absence of more specific data from the developer, Table A-5 is used to estimate a typical number of passenger seats for an aircraft with a seating capacity of about 115. This is equivalent to 230 passenger seats per LTO cycle. To estimate the number of passengers per LTO cycle, data provided by the developer on fraction of seating capacity utilization is useful. If this information is unavailable, information compiled for EPA by Thayer (August 1973) indicates that in 1970 approximately 47% of seats were filled. Hence, one should use Eq. (A27) or (A28) to estimate the number of passengers per LTO cycle:

$$P/LTO_{cycle} = \begin{array}{c}\text{Passenger}\\ \text{seats per}\\ \text{LTO cycle}\end{array} \cdot \begin{array}{c}\text{Fraction of}\\ \text{seating}\\ \text{utilized}\end{array} \qquad \text{(A27)}$$

where P/LTO_{cycle} is the number of passengers per LTO cycle.

Using data compiled for EPA (Thayer, August 1973) Eq. (A27) becomes

$$P/LTO_{cycle} = (230)\ (0.47) = 108. \qquad \text{(A28)}$$

Combining Eqs. (A18), (A20), (A23), (A24), (A25), one gets:

$$ADT = 0.75\ (\text{daily LTO cycles})\ (\text{P/LTO cycles})\ (1-F_t)\ [F_P + 2F_V] + 1.64\ (\text{number of employees}). \qquad \text{(A29)}$$

Once the average daily traffic volume is obtained from Eq. (A29), Eqs. (A12) to (A17) can be used to estimate traffic demand for the appropriate period of interest.

3. Shopping Centers

a. Input Parameters

This section provides guidance for air pollution control agencies, applicants, and consultants determining input parameters required for an analysis of CO concentrations in the immediate vicinity of a regional shopping center. A regional shopping center is defined as one with more than 300,000 square feet of gross leasable floor space and at least one major department store.

The major difference between regional and community or neighborhood shopping centers, other than size, is tenant mix. Generally, the tenant mix at the smaller shopping centers results in shorter visits and, therefore, smaller accumulations of vehicles within the parking lots. If it is possible to obtain an estimate of average daily trip generation rates and diurnal and seasonal usage patterns for a proposed community or neighborhood shopping center, the methodology described in this section could be applied to these smaller centers as well.

Table A-6 identifies key design and operating parameters that should be supplied by developers of shopping centers. Also listed are a number of optional parameters that should be obtained.

Among these parameters, physical configuration of the shopping center provides an indication of size and arrangement of vehicles within the center and offers a means for estimating the amount of running time required for a vehicle to enter the parking lot, move to a parking space, park, unpark, move to an exit, and exit. The parameters of number and capacity of exit/entrance gates are crucial since highest CO concentrations are likely to occur in the vicinity of the gates. If the volume demand at a gate begins to approach the gate's capacity, extensive queuing is likely. The probable result is a large increase in vehicle running times, leading, in turn, to greater emissions. Configuration and lane capacities of access roads are important, because they may influence the demand at exit/entrance gates and provide a determinant of gate capacity. In addition, access road capacity and the approach capacity at nearby intersections on access roads are prime considerations in determining whether CO levels will be high at the intersection as the result of extra traffic generated by the shopping center. Gross leasible floor space and tenant mix provide an indication of average trip duration and the number of parking spaces needed to accommodate demand and avoid congestion. Average daily trip generation rate, demand on access roads, peak demands, seasonal and diurnal demand patterns, and distribution of traffic among gates determine the volume of traffic utilizing the facility and passing key locations during any time of interest. When information on these volumes is

Table A-6

REGIONAL SHOPPING CENTER DESIGN AND OPERATING PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS

Parameters	Remarks
I Essential Parameters	
A. Design parameters	Should include such features as traffic lane locations, number of lanes at gates, design of gate approaches, and design intersection approaches on access roads.
Plans or blueprints of the shopping center and surroundings	
Number and capacity of exit/entrance gates	
Access road configurations and capacities (preferably by lane)	
Nearby intersection approach capacities	
Gross leasible floor space and tenant mix	
B. Operating parameters	
Seasonal and diurnal trip generation rate patterns and use patterns on access roads	
Angle of parking	Affects time needed to park and unpark a vehicle.
Average daily trip generation rate	Number of one-way trips per day. A trip to and from a shopping center represents two trips by this definition and thus counts each vehicle twice.
Traffic volume demand on access roads	Demand is the sum of traffic that is independent of the shopping center and shopping center traffic that has been properly apportioned among the roads on the surrounding road net.
Average visitor vehicle occupancy	
Fraction of visitors using mass transit	

Table A-6 (concluded)

Parameters	Remarks
I Essential Parameters (continued)	
B. Operating parameters (continued)	
Number of buses arriving and departing from the center during peak daily 1- and 8-hour use periods	
Green time to signal cycle ratio to each approach at nearby intersections	Needed to estimate approach capacities and queue lengths at approaches during the red phase of a signal.
Number of traffic signal cycles per hour at each intersection	Needed to estimate queue lengths at intersection approaches during the red phase of a signal.
II Additional Useful Parameters	
A. Design parameters	
Number of parking spaces available to visitors	
B. Operating parameters	
Peak 1- and 8-hour trip generation rates	
Highest 1- and 8-hour trip generation rates during periods of greatest nonshopping center traffic	
Distribution of traffic among gates	Includes information about numbers of left and right turns.
Distribution of operation modes (e.g. acceleration, deceleration) for a typical vehicle visiting a shopping center	Useful in obtaining good estimates for emission factors.

combined with capacity information, the amount of congestion can be estimated by volume demand-capacity ratios. Average vehicle occupancy and mass transit usage are important in relating number of customers to number of vehicles at the center.

b. Traffic Volume Demand

The are three ways in which traffic volume demand (V) can be estimated.

Use Peak 1-Hour and 8-Hour Trip Generation Rates--The first and most preferable approach is to use documented, peak 1-hour and 8-hour trip generation rates, in vehicles per hour (vph), provided by the developer for similar local facilities. Alternatively, if traffic on the surrounding road net is the prime consideration, periods in which the sum of shopping center traffic and other traffic is greatest would be of most interest. In locations where there is substantial traffic not related to the shopping center, the highest demand on the surrounding roads (attributable in part to the shopping center) may not occur when demand generated by the shopping center is greatest. For such cases, Clear (1974) has suggested using peak hourly traffic demands for 1- and 8-hour periods on the day of the year having the tenth highest daily demand. Such a procedure would avoid holidays and Saturdays in December, when unrelated traffic may be light.

Expected Average Daily Trip Generation Rate--The second-best approach is to use an expected average daily trip generation rate provided by the developer together with the seasonal and diurnal usage patterns for similar existing shopping centers in the area to estimate peak 1- and 8-hour traffic volume demand. This procedure is illustrated in Eqs. (A30) and (A31):

For a 1-hour peak period

$$V = (\text{ADT}) \begin{array}{l}\text{peak seasonal demand}\\ \text{adjustment factor}\end{array} \; \frac{\begin{array}{l}\text{peak hour demand at existing}\\ \text{nearby similar facilities}\end{array}}{\begin{array}{l}\text{peak seasonal daily demand at}\\ \text{existing nearby similar facilities}\end{array}} \;, \tag{A30}$$

where

ADT = average daily trip generation rate, vehicles per day,

V = Traffic volume demand, vph

For 8-hour peak periods

$$V = \frac{(\text{ADT}) \begin{array}{l}\text{peak seasonal demand}\\ \text{adjustment factor}\end{array} \; \dfrac{\begin{array}{l}\text{peak 8-hour demand at existing}\\ \text{nearby similar facilities}\end{array}}{\begin{array}{l}\text{peak seasonal daily demand at}\\ \text{existing nearby similar facilities}\end{array}}}{8} \tag{A31}$$

EPA Data for Regional Shopping Centers--If data from nearby existing facilities are unavailable, estimate peak 1-hour and 8-hour traffic volume demand from data compiled for EPA from a limited sample of regional shopping centers (Thayer, August 1973), as shown in Eqs. (A32) to (A35).

For the weekday peak one-hour volume, assumed to occur from 8 to 9 PM within 12 shopping days of Christmas (Thayer, August 1973),

$$V = 0.16 \text{ ADT} . \tag{A32}$$

For the weekday peak 8-hour volume, assumed to occur from 1 to 9 PM within 12 shopping days of Christmas (Thayer, August 1973),

$$V = 0.12 \text{ ADT} . \tag{A33}$$

For the Saturday peak hour volume, assumed to occur from 3 to 4 PM the Saturday before Christmas (Thayer, August 1973),

$$V = 0.24\ ADT\ . \qquad (A34)$$

For the Saturday peak 8-hour volume, assumed to occur from 10 to 6 PM the Saturday before Christmas (Thayer, August 1973),

$$V = 0.16\ ADT\ . \qquad (A35)$$

Table A-7 indicates ranges of average daily trip generation rates likely to be estimated for various sizes of regional shopping centers.

To note the impact of increasing the use of mass transit at the shopping center above the assumed usage in the developer's estimate of average daily trip generation rates, it is necessary to adjust the estimated volume demand. The adjusted volume demand could be estimated for one-hour periods as shown in Eq. (A36):

$$V' = V - \frac{(T)(P)}{(Avo)} + B\ , \qquad (A36)$$

where

V' = traffic volume demand adjusted for increased use of mass transit, vph,

T = number of mass transit passengers during the 1-hour period of interest,

P = fraction of passengers normally using a private automobile where mass transit is not available,

Avo = average number of passengers per automobile,

B = increase in number of buses during the hour due to increased use of mass transit, vph.

Table A-7

TRIP GENERATION RATES, AS A FUNCTION OF REGIONAL SHOPPING CENTER SIZE: DATA FROM THE COG* REPORT AND VALUES SUGGESTED FOR USE

Center Size (thousands of sq. ft.)	Number of Centers	Average-Day Trip Generation Rates (per 1000 square feet per day)					
		From the COG* Report			Suggested for Use		
		Median	Mean	Range	Median	Mean	Range
300-399	11	40	40	16-62	40	40	20-60
400-499	3	30	34	38-42	40	38	20-60
500-599	10	40	38	18-58	40	38	20-60
700-900	4	36	36	18-52	36	36	20-50
1000-1500	2	28	28	26-30	30	30	20-30
Total	30						

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*Source: Washington Metropolitan Council of Governments (July 1970).

Eq. (A37) applies to 8-hour periods:

$$V' = V - \frac{(T)(P)}{(8)(Avo)} + \frac{B}{8} \,, \qquad \text{(A37)}$$

where

T = number of passengers utilizing mass transit during 8-hour period of interest,

B = increase in number of buses during 8-hour period, vehicles per 8-hour period.

In addition to the overall traffic demand volume entering and exiting demand volumes should be estimated. The ADT includes trips going to or coming from the shopping center. In general, half of these trips enter and half leave on any day. The diurnal traffic pattern to and from the facility is a required parameter for performing an analysis of a shopping center. The diurnal pattern indicates the percentage of total trips attracted to, Pa, or generated by, Pg, the shopping center at particular hours of the day. Then the volume entering, Ve, or leaving, Vx, during any hour is given by Eqs. (A38) and (A39):

$$Ve = V' \frac{Pa}{Pg+Pa} \qquad (A38)$$

$$Vx = V' \frac{Pg}{Pg+Pa} \qquad (A39)$$

Ideally, the apportionment of entering and exiting traffic among gates should be provided by the developer, and should reflect the orientation of the shopping center with respect to the population center.

4. Sports Complexes

The purpose of this section is to assist air pollution control agencies and developers in estimating vehicular traffic and CO emissions associated with the operation of a sports complex.

Table A-8 presents the essential and desirable parameters that the developer should supply so that emissions associated with the operation of the sports complex can be estimated.

Total volume of automobiles utilizing parking facilities in the vicinity of the stadium can be estimated using Eq. (A40).

$$V' = \frac{(A)(P)}{(Avo)} , \qquad (A40)$$

Table A-8

SPORTS COMPLEX PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS

	Parameters	Remarks
I	Essential Parameters	
	Seating capacity	Assumed equivalent to peak attendance.
	Average attendance	Possible sources of information for this parameter include market surveys conducted for new teams or previous experience of established teams moving to new areas.
	Available parking spaces	This parameter should be broken up into three subtotals: Spaces available in off-street stadium-operated parking lots. Spaces in other off-street public and private parking lots. On-street spaces available within 1 km of the stadium property
	Number and capacity of parking lot gates	Should be on a set of plans or blueprints provided by the developer that present a schematic picture of access roads and complex dimensions and configuration.
	Surrounding street configurations, capacity, and speed limits (within 2 km of stadium)	Should be on a set of plans or blueprints provided by the developer that present a schematic picture of access roads and complex dimensions and configuration.

Table A-8 (concluded)

	Parameters	Remarks
I	Essential Parameters (cont.)	
	Estimated traffic volume on nearby streets not related to stadium activities	Should be solicited from local and state highway departments.
	Number of parking spaces allotted for buses	Should be available in parking lot plans.
	Stadium emptying time.	Time after end of an event by which all spectators have reached their parking spaces.
	Plans or blueprints of the stadium, parking lots, and surrounding access roads	Needed to prorate demand among gates and access roads and as an indicator of the complex's size.
	Green time to signal cycle ratio for each intersection	Needed to estimate traffic capacity at each intersection approach and to estimate resulting queue lengths at each approach.
	Number of signal cycles per hour at each intersection	Needed to estimate queue lengths at intersection approaches.
II	Desirable Parameters	
	Percentage of spectators arriving by automobile	
	Average vehicle occupancy	
	Number of buses	Estimate should be obtained from local transit authority or bus companies.
	Estimated order of preference for parking facilities	The developers' estimate of the order in which the three types of parking facilities are utilized by spectators.
	Vehicle spacing in queues	Tail pipe-to-tail pipe distance.
	Distribution of operation modes (e.g., acceleration, deceleration) for a typical vehicle visiting a sports complex.	Useful in obtaining estimates for emission factors.

where

V' = number of automobiles using parking facilities, vehicles,

A = attendance (for peak use periods, this can be considered equal to seating capacity),

P = percentage of spectators arriving by automobile,

Avo = average vehicle occupancy for automobiles.

Attendance is estimated from the stadium seating capacity. The percentage of spectators arriving by automobile and the average vehicle occupancy terms is ideally provided by the developer on the basis of the local situation and practices. If this is not possible, the following most commonly accepted design values can be used: 88% of spectators come by car with an average vehicle occupancy of 3.5 persons per automobile for football and 2.5 persons per automobile for baseball (Thayer, September 1973). For events other than football and baseball games and for facilities other than outdoor stadiums, the developer should be required to provide documented estimates of the percentage of spectators arriving by automobile and the average vehicle occupancy.

Automobile traffic volume, as determined by Eq. (A40), is the sum of vehicles parked in the stadium lots, on nearby streets, and in privately operated lots. Parking capacities for each type of facility within 1 km of the staduum site should be obtained from the developer to estimate localized high levels of CO. In addition, an order of parking preference should be indicated. Unless otherwise indicated, the order of preference should be: stadium lots, on-street parking, and privately owned parking lots (Thayer, September 1973). Under such an order of preference, it should be assumed that the stadium lots fill up first, followed by the on-street parking spaces and finally by privately operated lots, until the total traffic accommodated is equal to V'.

For purposes of assessing peak impacts on CO concentrations at exits or entrances or within parking lots, only analyses of off-street parking facilities are needed. On-street parking will be assessed in evaluating the imapct of the sports facility's operation on nearby access roads and at intersections.

For converting traffic volume to traffic volume demand, it is assumed that essentially all traffic will arrive or depart from the lots within an hour's time. Thus, even if traffic persists for less than an hour at a somewhat higher flow rate, the net effect will be the same in terms of emissions, providing the actual lot emptying and filling times are used in computing typical automobile running times.

Estimates for the number of buses (B) expected at an event should be obtained from the developer or promoter. If this estimate cannot be obtained directly, it can be made from the number of parking spaces allotted for buses. It seems reasonable to assume that the most heavily attended events are those at which the number of buses would approach the parking capacity for buses. Once the number of buses is determined, the same rationale as was applied for automobiles is used to obtain the bus volume demand (vph). The total volume demand from all arriving stadium traffic, which must be apportioned among access roads and added to traffic unrelated to the stadium using these roads, is estimated using Eq. (A41):

$$V_{TOT} = V' + B + V'_o \qquad \text{(A41)}$$

where

V_{TOT} = total demand from all stadium traffic before the event, vph,

V' = automobiles using off-street parking, vehicles

V'_o = automobiles using on-street parking, vehicles.

The demand for traffic to leave the stadium will be spread over the time it takes to empty the parking lot.

The total stadium traffic volume is the sum of off-street and on-street parking. This total stadium volume is converted to exiting and volume demand (for the purpose of estimating running time only) by using Eq. (A42):

$$V_{TOT} = 60 \frac{V' + B + V'_o}{PLET} \qquad (A42)$$

where

V_{TOT} = total demand from all stadium traffic after the event, vph,

V' = automobiles using off-street parking, vehicles,

B = buses present, vehicles,

60 = conversion factor from minute^{-1} to hour^{-1},

PLET = parking lot emptying time, minutes.

V_{TOT} is then apportioned among access roads and added to the nonstadium traffic on each road segment to obtain an estimate of the traffic volume demand for that road segment. This segment's total demand is divided by its capacity in the appropriate direction to obtain a demand-capacity ratio.

5. Municipal Parking Lots

a. Input Parameters

The general approach of this section's methodology is to require prospective developers to furnish certain information. A set of essential information and set of desirable information are identified. If a developer is unable to provide the second, optional, body of information, data compiled for a limited number of parking facilities in a report prepared for EPA (Thayer, October 1973) or estimation techniques described in this section will have to suffice.

Table A-9 presents information that should be provided by a developer to allow emissions from a parking facility to be estimated. The parking space capacity parameter is of greater importance for parking facilities than for other types of indirect sources; for these facilities the amount of related traffic depends more on this parameter than for other indirect sources. This dependence arises because, unlike other indirect sources, the parking facility is not the object of a trip. Rather, it serves to accommodate traffic representing individuals wanting to visit nearby attractions. Consequently, assuming there is a need for a parking facility, the peak amount of traffic utilizing it is largely determined by its parking space capacity.* The type of facility (garage or open lot) is primarily of interest in relating emissions to ambient CO concentrations. The type of parking utilized is of importance in estimating typical vehicle running times. Generally, running times are diminished with attendant parking. The physical layout and exit capacities are of importance in apportioning emissions throughout the facility and in determining running times when vehicle use rates approach exit/entrance capacity. The type of area in which the facility is located to a large extent determines the diurnal pattern of use rates and enables a more informed estimate of 1- and 8-hour peak use rates to be made. Direct estimates of peak 1-hour and 8-hour rates and base running times are also highly desirable. Data concerning the surrounding road network are of interest in determining whether the lot's presence is enough to cause an intolerable impact at nearby intersections.

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* This would not necessarily be true however, if the lot were a multi-purpose one, such as a sports stadium parking lot that were also used to park commuting traffic. In such cases, a more direct estimate of peak traffic volume demand would be needed.

Table A-9

PARKING FACILITY PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS

Parameters	Remarks
I Essential Parameters	
Parking space capacity	
Type of facility	Garage or open lot.
Type of parking	Attendant or self parking.
Number and capacity of exits/entrances and physical size and layout.	Available from plans or blueprints for the facility. In the case of attendant-operated facilities capacity is determined by the number of attendants on duty.
Type of area in which facility is located.	E.g., central shopping district, entertainment district, or fringe areas serving commuters.
Diurnal use rate patterns	
Traffic volume demand on access roads and its diurnal variation	
Access road capacities	
Number of parking attendants employed	
Green time to signal cycle ratio at nearby intersection approaches	Needed to estimate approach capacities at intersections and to estimate queue lengths when the signal is red.
Number of signal cycles per hour	Needed to estimate queue lengths during red signal phases.
Peak use rate	Vehicles per hour
Base running time	Seconds. Function of parking space configuration (e.g. 90^{o}, 45^{o}) and whether the facility uses self-or attendant-parking and physical configuration.

b. Traffic Volume Demand

Traffic volume demand or use rate is dependent on two factors:

- The number of parking spaces (i.e., parking space capacity available in the facility).
- The demographic and functional characteristics of the area served by the parking facility.

In the absence of indications to the contrary, it may be assumed that during periods of peak use the parking facility is used to capacity. Hence, the peak number of vehicles running during any period of interest is directly proportional to the facility's parking space capacity.

The demographic-functional characteristics of the area surrounding the parking facility determine the diurnal pattern (and hence the peak 1-hour and 8-hour volume demands) of the parking facility use rate. For example, the peak one-hour volume demand for a facility serving primarily commuting traffic might represent a very high percentage of the facility's parking space capacity, while the use rate averaged over an 8-hour period might be considerably lower. On the other hand, a parking facility located in a downtown commercial district used primarily for shopping might have a fairly steady use rate in which the 1- and 8-hour peak use rates agreed closely. The percentage of the facility's parking capacity that these use rates represented would depend on the type of shopping done and the length of stay it engendered.

There are two procedures that might be used to estimate peak use rated for a proposed parking facility if the developer has not provided an estimate of them, or if one wishes to check an estimate that is provided.

Observed Use Rates--Observe use rates at a nearby facility serving the same or a similar area. Then express these observed use rates (Ur) as a function of the existing parking facility's parking space capacity as shown in Eq. (A43) or Eq. (A44) for 1- and 8-hour sampling times, respectively:

$$Ur = \frac{\text{peak use rate, vph}}{\text{parking space capacity}} . \qquad (A43)$$

$$Ur = \left[\frac{1/8 \sum_{t=1}^{8} \text{use rate, vph}}{\text{parking space capacity}}\right]_{\substack{\text{peak contiguous}\\ \text{8-hour period}}} . \qquad (A44)$$

where

Ur = relative use rate, hour^{-1}.

Finally, combine the estimates obtained with Eq.. (A43) or (A44) with the proposed facility's parking space capacity to estimate the peak 1-hour or 8-hour traffic volume demand in Eq. (A45):

$$V = Ur\ (\text{parking space capacity}) , \qquad (A45)$$

where

V = peak traffic volume demand, vph.

Hourly Traffic Flow Rates--If there are no existing parking facilities in the vicinity of the proposed one, or if data concerning diurnal use rate patterns are unavailable, the following alternate technique for estimating peak 1-hour and 8-hour traffic volume demand can be used. Obtain hourly traffic flow rates on roads near the proposed facility from the local highway department or from traffic counts. Determine the total traffic flow on these roads during the operating hours of the proposed facility. Use Eqs. (A46) or (A47) to determine the peak 1-or 8-hour relative use (flow) rates, hour^{-1}:

$$\mathrm{Ur} = \frac{\text{peak hourly traffic flow rate, vph}}{\Sigma\ (\text{hourly flow rates during the hours of the proposed facility's operation, veh})} \qquad \text{(A46)}$$

$$\mathrm{Ur} = \left[\frac{1/8 \sum_{t=1}^{8} (\text{hourly traffic flow rate, vph})}{\Sigma\ (\text{hourly flow rates during hours of proposed facility's operation, veh})}\right]_{\text{peak contiguous 8-hour period}} \qquad \text{(A47)}$$

Finally, use Eq. (A45) to estimate peak 1-hour or 8-hour traffic volume demands.

In addition to the overall traffic demand volume, diurnal use rate patterns can be estimated.

Table A-10 presents the pertinent data on diurnal use rate patterns for a sample garage. It is assumed that the parking space capacity for the facility illustrated in the following example is 700 vehicles.

Table A-10

A SAMPLE PARKING GARAGE

Time of Day	Use Rate^{+} (Vehicles/Hour)	Relative Use Rate (hour^{-1})	Running Average 8-hour Relative Use Rate (hour^{-1})	
0800	17	0.024	0.003	
0900	62	0.089	0.014	
1000	147	0.21	0.040	*
1100	240	0.34	0.083	
1200	242	0.35	0.083	
1300	260	0.37	0.17	
1400	297	0.42*	0.23	
1500	266	0.38	0.27	
1600	283	0.40	0.32	
1700	223	0.32	0.35	
1800	131	0.19	0.35	
1900	14	0.02	0.31	

---

* = use rate is assumed to be the sum of vehicle entrances and exits during the hour.

+ = peak 1- and 8-hour relative use rates.

6. Amusement Parks

a. Input Parameters

Two key elements for emission control in the design of an amusement park become evident in assessing park impacts on emissions: (Axetell, November 1973).

- The entrance capacity must be sufficient to handle park arrival rates without creating long entrance queues.
- The number of parking spaces must be sufficient to accommodate the peak accumulation of vehicles in the lots, or an efficient operational means should route excess traffic to overflow parking areas.

The approach taken in this section is to identify a set of essential and a set of highly desirable amusement park design and operating parameters for assessing emissions. If it is not possible to obtain the optional information, the appropriate parameters can be estimated using information compiled for a number of existing amusement parks. (Axetell, November 1973) or by making best-judgement estimates on the basis of the parameters that the developer may supply. The essential and desirable parameters are then used to estimate peak traffic volume demand and accompanying running times for typical vehicles. This information, when combined with information concerning emission factors, enables estimates of vehicular emissions to be made.

Table A-11 presents the essential and the highly desirable parameters that should be provided by developers. In it, Peak Capacity is synonymous with peak total daily attendance. The anticipated diurnal arrival and departure pattern is a critical parameter in determining the number of vehicles running in a park's lots during the peak 1-8-hour use periods. Available parking spaces and gate capacities are extremely important in determining how congested a lot is likely to become with peak use rates. The percentage of crowd likely to arrive by automobile and the number of buses likely to be present are important parameters in that they enable better estimates of peak traffic volume

Table A-11

AMUSEMENT PARK PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS

Parameters	Remarks
I Essential Parameters	
Peak capacity	Maximum number of visitors that can be accomodated daily with the existing park configuration.
Anticipated diurnal arrival and departure patterns	Fraction that entering and leaving vehicles comprise of the total daily arrivals during each hour of the park's operation on days with capacity crowds.
Available visitor and employee parking spaces, physical layout, and dimensions of the amusement park	Needed to help estimate running times and identify locations with heaviest congestion.
Number and percentage of employees arriving by automobile	
Number of buses expected and fraction of visitors using mass transit	
Gate capacities	Number of vehicles that can be accomodated through each entrance (exit) per unit time.
Traffic volume demand and capacities for access roads and at intersection approaches	Needed to determine the impact of the park at nearby approaches.
Base running time (BRT)	Total time a typical vehicle is likely to spend with the engine on while arriving and leaving the park during periods with little or no congestion.
Average vehicle occupancy	The average number of visitors per vehicle expected during periods with capacity crowds.
Estimated ratio of people in the Park at the most crowded time of day to total daily admission tickets sold.	
Estimated time required for vehicles to move to auxiliary lots when the parking capacity in the main lot is exceeded.	
Distribution of vehicles through exit/entrance gates	

demand to be made, given crowd projections. Average vehicle occupancy is a highly desirable parameter because it provides a means for estimating traffic volume from crowd projections. Ratio of people in the park to total daily admission ticket sales is also desirable because it facilitates estimating peak occupancy in the parking lot. Base running time is important in estimating typical emission levels with little congestion, while the excess time required for traffic to move to auxiliary lots when main lot capacity is exceeded may be of interest for periods of peak use. Knowing the distribution of traffic demand among exit/entrance gates and access roads is essential for determining whether congestion is likely to occur at these key locations.

b. Traffic Volume Demand

For any number of hours t, the traffic volume demand can be estimated suing Eq. (A48):

$$V = \frac{(P)\ (Pv)\ (Ap)}{Avo} + Pv\ B + \frac{(Pae)\ (E)\ (Pemp)}{Avoe} \qquad (A48)$$

where P = fraction of the crowd arriving by automobile,

Pv = fraction that entering and leaving vehicles comprise, during time period t, of the total daily vehicle arrivals,

Ap = estimated peak total daily admission tickets sold (peak capacity parameter in Table A-11),

Avo = average visitor automobile occupancy

Avoe = average employee vehicle occupancy,

B = number of buses arriving and leaving during an average day,

Pemp = average hourly fraction of employees arriving and leaving during time period t, estimated from diurnal arrival-departure patterns for employees, V = traffic volume demand, vph,

E = number of park employees,

Pae = fraction of employees arriving by automobile.

Estimates of the parameters Ap, B, E, P, and Pae must be supplied by the developer. The parameters Pv and Pemp are derivable from the estimates of diurnal arrival and departure patterns required from the developer, as shown by Eqs. (A49) and (A50):

$$Pv = \frac{(\text{arrivals + departures})_t}{(\text{arrivals on the peak day})\ (t)} \qquad \text{(A49)}$$

or

$$Pv = \frac{(\text{fraction of total daily arrivals})\ t \ + \ (\text{fraction of total daily departures})\ t}{t} \qquad \text{(A50)}$$

where t is the number of hours in time period t, hours, Pemp can be estimated with Eq. (A51):

$$Pemp = \frac{(\#\text{ of employee arrivals} + \#\text{ of employee departures})}{(\#\text{ of employees})\ (t)}\, t \qquad \text{(A51)}$$

Average visitor vehicle occupancy (Avo) is a highly desirable parameter to obtain from the developer. However, if this is not possible, past observations (Axetell, November, 1973) have indicated a range of about 3.2 to 4.0 persons per vehicle with a mean of 3.6. Therefore, in the absence of better information, a 3.6 Avo should be assumed. Unless there are indications to the contrary, a 1.3 average vehicle occupancy for employees, Avoe can be assumed.

For consistency, the diurnal arrival and departure patterns provided by the developer should be checked against the ratio of people in the park at the most crowded time of day to total daily admission tickets sold (R). While parameters of anticipated diurnal arrival and departure patterns are required, they may be somewhat difficult to estimate. The information in Table A-12 is based on observations at existing amusement parks (Axetell, November, 1973) and is provided as a guide.

Table A-12

DIURNAL USE PATTERN PARAMETERS OBSERVED AT SELECTED AMUSEMENT PARKS

Time Period (hours)	Pv	R
1	0.25	0.85
8	0.15	0.73

R = ratio of people in the park at the most crowded time of day to total daily ticket sales. (November, 1973).

The parameter V_i (traffic volume demand at gate i, v/hr) is obtained by apportioning the entering or exiting traffic volume demand among entering or exit gates. Ideally, the apportionment of entering and exiting traffic is provided by the developer, and is based on the orientation of the amusement park with respect to population centers. Entering traffic is apportioned directionally on the access road, and the traffic in each direction is set equal to V_i. The excess running time spent entering entrance i is the mean value of the excess time spent in each direction on the access road. If the apportionment is not provided, two procedures to estimate it can be used.

- Apportion exiting and entering traffic according to access road capacities.
- Apportion traffic according to gate capacities if access capacities are unknown, or if more than one gate empties onto the same access road.

If it is not possible to estimate exiting and entering traffic directly, estimates can be obtained from a limited data base compiled for EPA (Axetell, November, 1973). In examining diurnal arrival and departure rates for existing amusement parks comprising the EPA data base, it has been noted that the peak hourly demand (as determined by arrivals and departures) always occurs durng periods in which traffic flow is either totally dominated by arrivals or by departures. Similarly, for 8-hour periods in which peak average hourly demand generated by visitors was observed, one mode outnumbered the other by a ratio of about 3:1. On the basis of these observations, for one-hour periods in which peak demand occurs, all visitor traffic is assumed to be either entering or exiting. Similarly, for an 8-hour sampling period in which peak traffic volume demand occurs, it is assumed that 75% of the demand generated by visitors is trying to exit or enter. Thus, in the case of parks with more than one exit/entrance for peak 1-hour periods, the total peak traffic volume demand must be apportioned among the exit (or entrance) gates.

For 8-hour sampling periods, 75% of the average hourly demand (over the 8-hour period) should be apportioned among the entrances (exits), and the remaining 25% among the exits (or entrances).

7. Recreational Areas

a. Input Parameters

Recreational area design and operating parameters fall into two categories: those essential to and those highly desirable for estimating emissions; both should be provided by the developer. These parameters are used to estimate vehicle running times and peak traffic demand which, in conjunction with vehicle emissions factors, are used to estimate vehicular emissions.

Two key elements for emission control in the design of a recreational area become evident in assessing recreational area impacts on emissions: (Axetell, November, 1973)

- The entrance capacity must be sufficient to handle peak arrival rates without creating long entrance queues.
- The number of parking spaces must be sufficient to accommodate the peak accumulation of vehicles in the lots, or an efficient operational means should route excess traffic to overflow parking areas.

Table A-13 presents the essential and the highly desirable recreational

Table A-13

RECREATIONAL AREA PARAMETERS NEEDED FOR AIR QUALITY IMPACT ANALYSIS

I Essential Parameters

Parameter	Remarks
Peak capacity.	Maximum number of visitors that can be accommodated daily with the existing park facilities.
Anticipated diurnal arrival and departure patterns.	Percentage of total arrivals anticipated for entering and leaving vehicles during each hour of the park's operation on days with capacity crowds.
Available visitor parking spaces.	
Physical layout and dimensions of the recreational area's parking facilities.	Needed to help estimate running times and identify location with heaviest congestion.
Percentage of crowd arriving by automobile.	
Number of buses expected.	
Number of camper type vehicles expected.	
Gate capacities.	Number of vehicles that can be accommodated through each entrance (exit) per unit time.
Green time to signal cycle ratio at nearby intersection approaches.	Needed to estimate intersection approach capacities and queue lengths on approaches during the red phase.
Number of signal cycles per hour at each nearby intersection.	Needed to estimate queue lengths on each approach to each intersection during the red phase for each approach.

Table A-13 (Continued)

Parameter	Remarks
I Essential Parameters (continued)	
Traffic volume demand and capacities for access roads and at nearby intersection approaches.	Needed to estimate the recreational area's impact at nearby intersection approaches.
II Highly Desirable Parameters	
Base running time (Brt).	Total time a typical vehicle spends with the engine on while arriving and leaving the park during periods with little or no congestion.
Average vehicle occupancy.	The average number of visitors per vehicle during periods with capacity crowds.
Estimated ratio of people in the park at the most crowded time of the day to total daily attendance.	
Estimated time required for vehicles to move to auxiliary lots when the parking main lot capacity is exceeded.	
Distribution of vehicles through exit/entrance gates.	

area parameters that the developer or future administrator should provide.

Peak capacity is an indicator of the maximum number of vehicles likely to utilize the area's parking lots during a day. Diurnal arrival and departure patterns are critical parameters for determining the number of vehicles running in a park's lot during the peak 1- and 8-hour use periods. Knowledge of available parking spaces and gate capacities is extremely important in determining how congested a lot is likely to become with peak use rates. Percentage of crowd likely to arrive by automobile, number of camper type vehicles, and number of buses likely to be present are important parameters in that they enable better estimates of peak traffic volume demand to be made, given crowd projections. Average vehicle occupancy is a highly desirable parameter because it provides a means for estimating traffic volume from crowd projections. Ratio of people in the park to total daily attendance is also desirable because it facilitates an estimate of peak occupancy in the parking lot. Base running time is of importance because it is needed to estimate typical emission levels with little congestion, while the excess time required for traffic to move to auxiliary lots when main lot capacity is exceeded may be of interest during periods of peak use. Distribution of traffic volume demand among exit/entrance gates and access roads is needed in estimating whether congestion is likely to occur at these key locations.

b. Traffic Volume Demand

For any number of hours, t, the traffic volume demand can be estimated using Eq. (A-52).

$$V = \frac{P\ PvAp}{(Avo)} + Phv\ B \qquad \text{(A-52)}$$

where P = fraction of the crowd arriving by automobile,

Pv = average hourly fraction of total daily automobile and camper arrivals and departures during time period t,

Phv = average hourly fraction of total daily heavy-duty vehicle arrivals and departures during time period t,

Ap = estimated peak total daily admission tickets sold (peak capacity parameter in Table A-15),

Avo = average automobile occupancy for visitors,

V = traffic volume demand, vph,

Bh = heavy-duty traffic demand, vph.

Estimates of the parameters Ap, Bh, and P must be supplied by the developer and would be used directly in Eq. (A-52). The parameter Pv is derivable from the estimates of diurnal arrival and departure patterns required from the developer, as shown by Eqs. (A-53) and (A-54).

$$\text{Pv} = \frac{(\text{arrivals + departures})_t}{(\text{arrivals on the peak day})\ (t)} \tag{A-53}$$

$$\text{Pv} = \frac{\begin{array}{c}\text{(fraction of total} \\ \text{daily arrivals)}_t\end{array} + \begin{array}{c}\text{(fraction of total} \\ \text{daily departures)}_t\end{array}}{t} \tag{A-54}$$

where t = number of hours in time period t, hours.

Unless there are explicit indications to the contrary, Phv can be assumed equal to Pv. Average visitor vehicle occupancy (Avo) is a highly desirable parameter to obtain from the developer. However, if this is not possible, past observations (Axetell, November, 1973) have indicated typical automobile occupancy of about 3.5 persons per vehicle. Therefore, in the absence of better information, a 3.5 Avo should be assumed.

While the anticipated diurnal arrival and departure patterns are required data, they may be somewhat difficult to estimate. The information in Table A-14 is based on observations at existing recreational areas, (Axetell, November, 1973) and is provided as a guide.

Table A-14

DIURNAL USE PATTERN PARAMETERS OBSERVED AT SELECTED RECREATIONAL AREAS

Time Period	P_v*
1 hour	0.18
8 hours	0.16

*P_v = Average hourly fraction of total daily automobile and camper arrivals and departures during time period t.

Source: Axetell (November, 1973)

The apportionment of entering and exiting traffic, based on the orientation of the recreational area with respect to population centers. If the apportionment is not provided, one of two apportioning procedures can be followed:

- Apportion exiting and entering traffic according to access road capacities.
- If access road capacities are unknown, or if more than one gate empties onto the same access road, apportion traffic according to gate capacities.

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Appendix B

METHODS OF ESTIMATING ROADWAY CAPACITY

Several publications deal with the theoretical computation of capacity of a road (Highway Capacity Manual, 1965; Liesch, 1967; Kennedy, 1973, Reilly, 1975; Traffic Institute, Northwestern University, 1967). The user may want to consult one or more of these references when a complex capacity analysis is required or if he finds the analysis of this appendix too restrictive. For example, a complex analysis might be required at an intersection with separate signal phases for turning movements, overlapping left-turn and through signal phases, an unusual peaking of arrivals during the peak hour, or large pedestrian volumes that might influence signal timing. However, in most cases a complex capacity analysis is unnecessary and the methods of this appendix can be used.

The methodology developed in this appendix is conservative in that it usually underestimates capacity.

Four types of capacity analysis are described in this appendix: (1) free flow, (2) signalized intersections, (3) four-way stop intersections, and (4) two-way stop intersections.

1. Free Flow Capacity

The Highway Capacity Manual (1965) gives the following maximum uninterrupted flow capacities under ideal conditions for various types of roadways:

Highway Type	Capacity (vph)
Multilane	2,000 per lane
Two-lane, two-way	2,000 total (both directions)
Three-lane, two-way	4,000 total (both directions)

A multilane facility here is one with greater than two lanes in either direction, limited or free access.

The capacity, C, of a multilane roadway is computed using the following equation:

$$C = 2000\ M W_f\ T; \qquad (B1)$$

the capacity for one direction of a two-lane roadway is computed using the equation:

$$C = 1000\ W_f\ T \qquad (B2)$$

where

M = number of lanes moving in one direction
Wf = adjustment factor for lane width from Table B-1
T = truck factor from Table B-2.

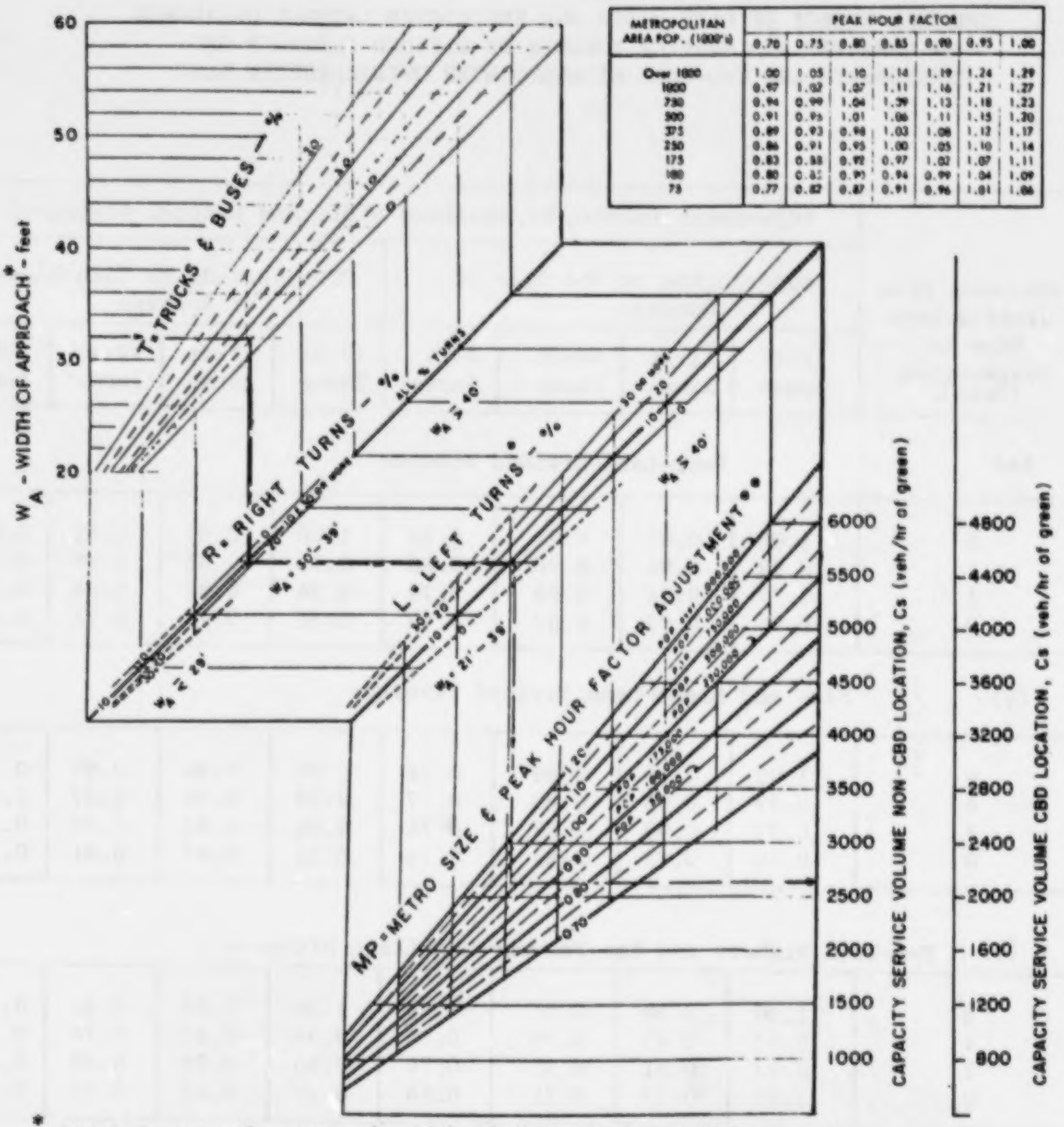
2. Signalized Intersection Capacity

Any at-grade intersection approach has a capacity that represents the maximum number of vheicles that can be accommodated given the particular geometrics, environment, and traffic characteristics and controls. The capacity service volume of an intersection approach is the maximum number of vehicles that can pass through the intersection during one hour of green time. The number of vehicles that can clear the intersection from an approach during one hour of elapsed time can be calculated by multiplying the fraction of the total cycle time that the signal is green by the capacity service volume.

The capacity service volume in vehicles per hour of green is determined using the nomograph, Figure B-1. The user must know the percentage of trucks and buses, left turners, right turners, the location within the metropolitan area, the metropolitan area size, and whether the intersection is located in the CBD (Central Business District) or non-CBD. The nomograph provides a solution for a two-way urban street with parking. The solution to this type of intersection is the most conservative estimate of capacity in the Highway Capacity Manual. If a street has no parking within 250 feet of the intersection, 8 feet can be added to the curb-to-center line width (Wa) and a conservative solution will still result from use of the nomograph

TABLE 8 MP = METROPOLITAN SIZE AND PEAK HOUR FACTOR ADJUSTMENT

METROPOLITAN AREA POP. (1000's)	PEAK HOUR FACTOR						
	0.70	0.75	0.80	0.85	0.90	0.95	1.00
Over 1000	1.00	1.05	1.10	1.14	1.19	1.24	1.29
1000	0.97	1.02	1.07	1.11	1.16	1.21	1.27
750	0.94	0.99	1.04	1.09	1.13	1.18	1.23
500	0.91	0.96	1.01	1.06	1.11	1.15	1.20
375	0.89	0.93	0.98	1.03	1.08	1.12	1.17
250	0.86	0.91	0.95	1.00	1.05	1.10	1.14
175	0.83	0.88	0.92	0.97	1.02	1.07	1.11
100	0.80	0.85	0.90	0.94	0.99	1.04	1.09
75	0.77	0.82	0.87	0.91	0.96	1.01	1.06



* Add 8 feet to the approach width if there is no parking.

** When the peak hour factor is known, use table above to determine MP; when peak hour factor is not known use population directly.

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FIGURE B-1 SERVICE VOLUME OF A SIGNALIZED INTERSECTION APPROACH

Table B-1

COMBINED EFFECT OF LANE WIDTH AND RESTRICTED LATERAL CLEARANCE ON CAPACITY AND SERVICE VOLUMES OF DIVIDED FREEWAYS AND EXPRESSWAYS AND TWO-LANE HIGHWAYS WITH UNINTERRUPTED FLOW

Distance from Traffic Lane Edge to Obstruction (Feet)	Adjustment Factor, Wf, for Lane Width and Lateral Clearance							
	Obstruction on One Side of Roadway				Obstructions on Both Sides of Roadway			
	12-ft lanes	11-ft lanes	10-ft lanes	9-ft lanes	12-ft lanes	11-ft lanes	10-ft lanes	9-ft lanes
(a)	Four-Lane Divided Freeway							
6	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
4	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
2	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
0	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
(b)	Six- and Eight-Lane Divided Freeways							
6	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
4	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
2	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
0	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70
(c)	Two-Lane Highway and Non-Freeway Multilane Highways							
6	1.00	0.88	0.81	0.76	1.00	0.88	0.81	0.76
4	0.97	0.85	0.79	0.74	0.94	0.83	0.76	0.71
2	0.93	0.81	0.75	0.70	0.85	0.75	0.69	0.65
0	0.88	0.77	0.71	0.66	0.76	0.67	0.62	0.58

Table B-2

AVERAGE CAPACITY ADJUSTMENT FACTORS FOR TRUCKS

Percentage of Trucks (%)	Factor, T, For All Levels of Service		
	Level Terrain	Rolling Terrain	Mountainous Terrain
	Freeways and Expressways		
1	0.99	0.97	0.93
2	0.98	0.94	0.88
3	0.97	0.92	0.83
4	0.96	0.89	0.78
5	0.95	0.87	0.74
6	0.94	0.85	0.70
7	0.93	0.83	0.67
8	0.93	0.81	0.64
9	0.92	0.79	0.61
10	0.91	0.77	0.59
11	0.89	0.74	0.54
14	0.88	0.70	0.51
16	0.86	0.68	0.47
18	0.85	0.65	0.44
20	0.83	0.63	0.42
Two-Lane Highways and Non-Freeway Multi-Lane Highways			
1	0.99	0.96	0.90
2	0.98	0.93	0.82
3	0.97	0.89	0.75
4	0.96	0.86	0.69
5	0.95	0.83	0.65
6	0.94	0.81	0.60
7	0.93	0.78	0.57
8	0.93	0.76	0.53
9	0.92	0.74	0.50
10	0.91	0.71	0.48
12	0.89	0.68	0.43
14	0.88	0.64	0.39
16	0.86	0.61	0.36
18	0.85	0.58	0.34
20	0.83	0.56	0.31

(see Reilly, 1975, for discussion of the influence of no-parking distance on capacity).

The approach capacity, C, is determined by multiplying the capacity service volume by the G/Cy ratio for the approach. The green phase time, G, and cycle time, Cy, are determined by Webster's "Traffic Signal Settings" (1958). Webster's equations provide enough cycle time for the vehicle demand volume to proceed through the intersection; however, the user must ensure that G for each phase is long enough for pedestrian crossings. To determine Webster's optimum cycle length, the critical or maximum volume to capacity service volumes must be determined for each phase. The maximum volume to capacity service volume for phase j is written as

$$\underset{i}{\text{Max}}\ [V_{i,j}/Cs_{i,j}]$$

and refers to those V/Cs ratios for approachs or lanes of approaches on which traffic moves on phase j. For example, if approaches 1 (i=1) and 3 (i=3) to an intersection move on phase 1 (j=1) of a signal, then

$$\underset{i}{\text{Max}}\ [V_{i,j}/Cs_{i,j}] = \text{Max}\ [V_{1,1}/Cs_{1,1},\ V_{3,1}/Cs_{3,1}]\ ,$$

and if phase 1 above is a left turn phase, and phase 2 controls through and right turning traffic on approaches 1 and 3, then

$$\underset{i}{\text{Max}}\ [V_{i,j}/Cs_{i,j}] = \text{Max}\ [V_{1,2}/Cs_{1,2},\ V_{3,2}/Cs_{3,2}]\ .$$

In the above example, $V_{1,1}$ is the left turn volume demand for approach 1 and $Cs_{1,1}$ is the capacity service volume for left turning traffic from approach 1; similarly, $V_{1,2}$ is the through and right turning volume demand for approach 1 and $Cs_{1,2}$ is the capacity service volume for through and right turning traffic on approach 1. The optimum signal cycle length is determined using the following equation:

$$Cy = \frac{(9\ Np + 5)}{1 - \sum\limits_{\text{all } j} \max\limits_{i} [V_{i,j}/Cs_{i,j}]} \qquad \text{(B3)}$$

where

Np is the number of amber intervals per signal cycle during which there is no simultaneous green phase.

9 is a weighted lost time factor which assumes three seconds of amber time and three seconds of start up time.

$V_{i,j}$ is the volume on the i^{th} approach that moves during the j^{th} signal green phase.

$Cs_{i,j}$ is the capacity per hour of green to vehicles on the i^{th} approach moving during the j^{th} signal phase.

The green phase length is a fraction of the signal cycle time minus the total amber time. A 3-second amber time is assumed for all green phases. (A 3-second amber time is usually adequate for roadways with a speed limit less than or equal to 35 mph. A 4-second amber time is applicable for speeds of 35 to 50 mph, and a 5-second amber time is applicable beyond 50 mph.) The green phase length of phase j is given by the following equation:

$$G_j = Cy \frac{\max\limits_{i} [V_{i,j}/Cs_{i,j}]}{\sum\limits_{\text{all } j} \max\limits_{i} [V_{i,j}/Cs_{i,j}]} - 3$$

where

$\max\limits_{i} [V_{i,j}/Cs_{i,j}]$ is the maximum V/Cs ratio on all approaches i moving on green phase j.

3 is an assumed 3-second amber time.

$\sum\limits_{\text{all } j} [V_{i,j}/Cs_{i,j}]$ is the sum of the V/Cs ratios that control the green phase durations.

The approach capacity is found by multiplying the approach capacity service volume by the appropriate green to cycle ratio and summing for all applicable phases. Since an approach is considered in the guidelines to be one direction of flow into an intersection, separate left turning, through, and right turning signal phases affecting one approach should have their capacities added together to determine the total capacity of an approach. The capacity of an approach is given as follows:

$$C_i = \sum_j Cs_{i,j} \; Gj/Cy$$

where j are those green signal phases that allow traffic to move on intersection approach i.

Worksheet B presents a format for solutions to the equations of this appendix. The worksheet provides space for entries as if two signal phases controlled each approach. Thus, a left turn green phase as well as a through phase can be designated for each approach. Most types of signal controllers can be handled with the worksheet layout (see examples in Chapter IV).

Examples of controllers for which the worksheet is applicable are three and four phase controllers, which have a preceding left turn green indication, three and four phase controllers with one or two left turn phases, and eight phase controllers with possible overlapping left turn and through phases (i.e., multiple phases) on all opposing approaches. Usually the signal cycle time is the sum of the green phase times and amber times for all phases. Where overlapping phases occur, the cycle time is the sum of the left turn and through phases plus amber time when there is no simultaneous green indicator. The capacity of an approach is the sum of the capacities for each through or turning movement on the approach.

3. Two-Way Stop or Yield Capacity

The analysis of a two-way stop or yield approach is difficult because the capacity depends on traffic flow on the major cross street and on individual driver decisions on when to proceed. The formula used to

WORKSHEET B--CAPACITY ANALYSIS (see instructions following)

Step	Symbol	Input/Units	
1	i	Road segment (or approch) designation	____ ____ ____ ____
2		Free flow capacity computation:	
2.1	M_i	Number of lanes	____ ____ ____ ____
2.2	W_f	Adjustment for lane width (Table B-1)	____ ____ ____ ____
2.3	T_i	Adjustment for trucks (Table B-2)	____ ____ ____ ____
2.4	C_i	Free flow capacity	____ ____ ____ ____
3		Signalized intersection capacity:	
3.1	j	Green signal phase identification	__ __ __ __ __ __ __ __ __
3.2	Wa_i	Approach width with parking (ft)	__ __ __ __ __ __ __ __ __
3.3		Percent right turners	__ __ __ __ __ __ __ __ __
3.4		Percent left turners	__ __ __ __ __ __ __ __ __
3.5		Metropolitan area size	
3.6	$Cs_{i,j}$	Capacity service volume (vph of green)	__ __ __ __ __ __ __ __ __
4		Signalized intersection green phase and cycle length:	
4.1	$V_{i,j}$	Demand volume for approach and phase	__ __ __ __ __ __ __ __ __
4.2	$V_{i,j}/Cs_{i,j}$	Volume to green capacity ratio	__ __ __ __ __ __ __ __ __
4.3	approx G/Cy	Approximate G/Cy	__ __ __ __ __ __ __ __ __
4.4	$\sum_{j} \max(V_{i,j}/Cs_{i,j})$	Sum of the maximum V/C ratios for each signal phase	____
4.5	Cy	Signal cycle time (sec)	____
4.6	Gj	Green phase length	__ __ __ __ __ __ __ __ __
4.7	Gj/Cy	Green phase to cycle time ratio	__ __ __ __ __ __ __ __ __
4.8	$C_{i,j}$	Capacity for approach i phase i	__ __ __ __ __ __ __ __ __
5		Two-way stop, two-way yield or uncontrolled intersection:	
5.1	$V_m + V_n$	Major street two-way volume	____ ____
5.2	C_i	Cross street capacity	____ ____ ____ ____
6		Four-way stop intersections:	
6.1	V_i	Approach volume	____ ____ ____ ____
6.2	Spi	Demand split on cross streets	____ ____
6.3	C_i	Capacity of approach	____ ____ ____ ____
7	C_i	Approach capacity $\sum_{j} C_{i,j}$ 5.2 for a four-way stop or 6.3 for a two-way stop	____ ____ ____ ____

INSTRUCTIONS FOR COMPLETING WORKSHEET B

Line	Instructions
1	Enter the segment or approach identification from Worksheet 1.
2.1	Enter the number of lanes for *one direction of through flow*. If the road is two-lane, two-way then enter 0.5; if three-lane two-way, then enter 1.
2.2	Enter an adjustment factor for lane width from Table B-1.
2.3	Enter an adjustment for trucks from Table B-2.
2.4	Find the product of 2000 times line 2.1 times line 2.2 times line 2.3.
3.1	Assign an index to each possible green phase. Enter the index above the turning movement controlled. Approaches controlled by multiple phases should have an index entered for each possible phase.
3.2	Enter the width of the approach for each phase (include turn lanes). Add 8 ft if there is no parking (usually the case with left turn lanes).
3.3	Enter the percentage of right turns from Worksheet 1.
3.4	Enter the percentage of left turns from Worksheet 1 except where there is no opposing traffic flow, enter zero.
3.5	Enter the estimated metropolitan population from Worksheet 1.
3.6	Determine the capacity service volume from Figure B-1. Use the CBD scale when determining left turn lane capacity.
4.1	Find approach volume from Worheet 1. For each possible green phase enter a demand volume. For a left turn phase, it is the percent of left turns times the approach volume. For a through and right turn phase it is the approach volume minus the left turning volume. Where multiple phases control one flow, the volume served on the earlier phase must be determined before the demand for the subsequent phase can be determined. This involves using an approximate G/Cy ratio from line 4.3 below for the earlier multiple phase and then determining demand volume for the later multiple phase.
4.2	Divide line 4.1 by line 3.6 to obtain the ratio of volume to capacity service volume. The possible signal phases identified on line 3.1 that are active due to respective approach V/Cs ratios can now be determined. Circle the green phase indexes on line 3.1 for the phases with the largest V/C ratios, which determine the duration of the green phase.

Line	Instructions
4.3	Make a first approximation of G/Cy ratios for each green phase circles on line 3.1. This is (a) the V/Cs ratio from line 4.2; (b) if the green phase is of fixed or maximum duration, it is the green interval divided by the expected cycle length; or (c) if there is multiple phasing, a new V/Cs ratio (G/Cy estimate) must be determined for the second multiple phase based on the volume that remains to be processed after the first phase is complete.
4.4	Sum the maximum values from line 4.3.
4.5	Multiply by 9 the number of phases for which an amber interval was given during which there was no simultaneous green phase, add 5, and divide by the difference of 1 minus line 4.4. Enter 180 if the value is greater than 180, and enter 60 if the value is less than 60.
4.6	Multiply the maximum value for each phase from line 4.3 by line 4.5, divide by line 4.4, and from the result subtract 3. If less than 10 seconds, enter 10 and adjust other phases accordingly.
4.7	Divide line 4.6 by line 4.5. If line 4.7 is less than line 4.3, the appropriate green phase length must be increased or a less restrictive capacity analysis than is presented in this appendix must be undertaken.
4.8	Multiply line 4.7 times line 3.6.
5.1	Add approach volumes from opposing approaches from Table A-1.
5.2	Determine cross street approach capacity using line 5.1 to enter Figure B-2.
6.1	Enter approach volumes on each intersection approach from Table B-1.
6.2	Divide the maximum approach volume by the sum of the maximum volumes on one approach of each cross street.
6.3	Enter Figure B-3 with the ratio from 6.2 and determine the capacity of each approach on each street. (Capacities on each street are equal.)
7	Enter the approach capacity. Sum line 4.8 for each phase of a signalized intersection, or for unsignalized intersections line 5.2 for two-way stops, or line 6.3 for four-way stops.

compute capacity assumes a Poisson distribution of arrivals on the major cross street. The minimum gap acceptable to the side-street driver is assumed to be 8 seconds, the second and subsequent side-street vehicles can follow if there is an additional 8-second gap. The 8-second gap acceptance is conservative. When other information exists, the user may wish to evaluate capacity using other methods than presented in this screening procedure capacity analysis. (See Drew, 1968; Gazis, 1974; Kennedy, 1973; Johannessen, 1975).

Figure B-2 has been developed for the user to determine approach capacity on the minor cross street based on major cross street, two-way flow. The figure is entered with the total volume on the major cross street $(V_m + V_n)$ and an approach capacity for the minor cross street is determined. The formula used to determine capacity in Figure B-2 is as follows:

$$C_i = \frac{(V_m + V_n)\ e^{-8(V_m + V_n)/3600}}{1 - e^{-8(V_m + V_n)/3600}}$$

where

V_m is one directional volume on the major street in vph.

V_n is the other directional volume on the major street in vph.

$V_m + V_n$ is a two-way volume on the major street.

The capacity on the major street is assumed to be equal to the free flow capacity on that street. It is also assumed that no vehicles on the major street stop at the intersection.

4. Four-Way Stop Capacity

Four-way stop controls produce more predictable traffic operation than do two-way stop controls because all legs have equal priority. Under capacity conditions, a regular discharge of traffic occurs when the flow on the two cross streets is approximately equal. A nomograph (Figure B-3), based on the tables presented in the Highway Capacity Manual

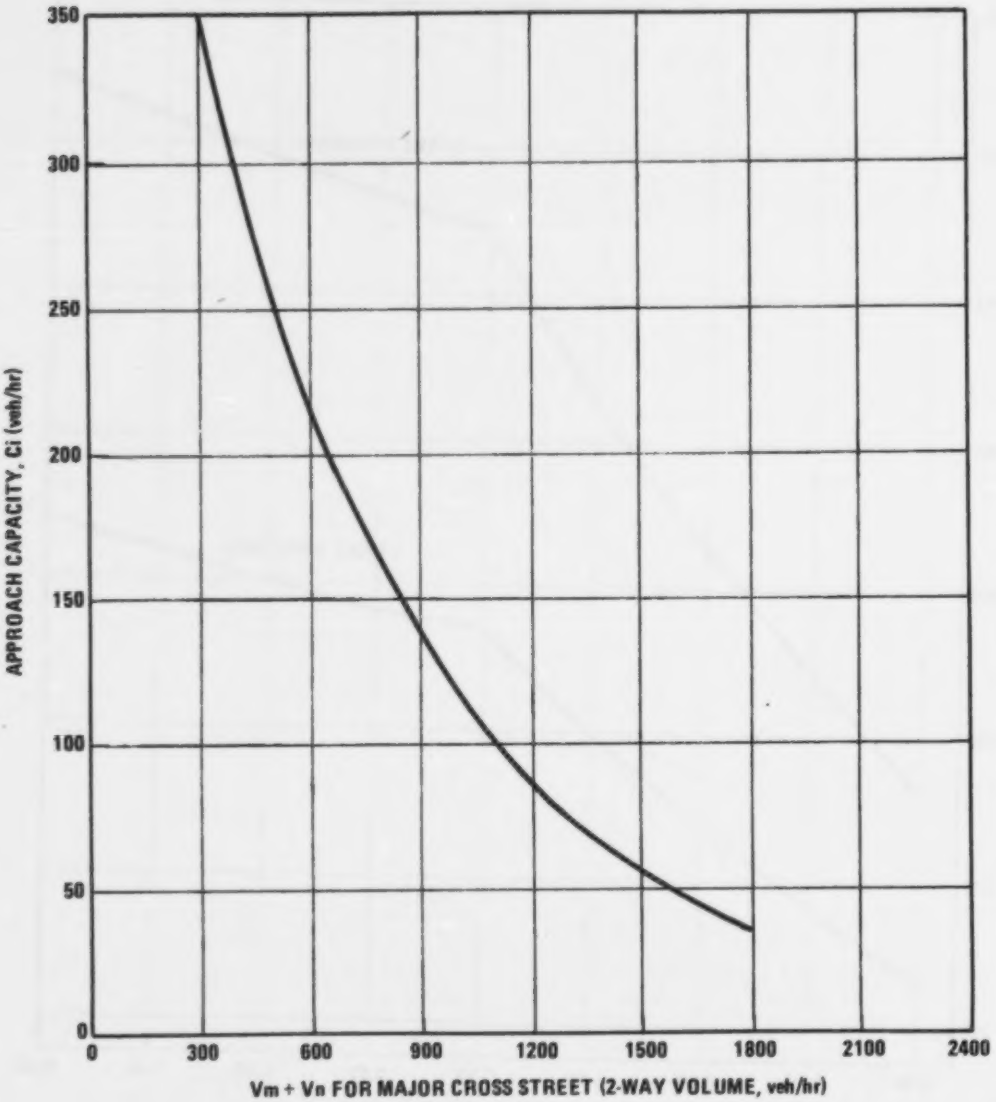

Figure B-2. Capacity at a two-way stop.

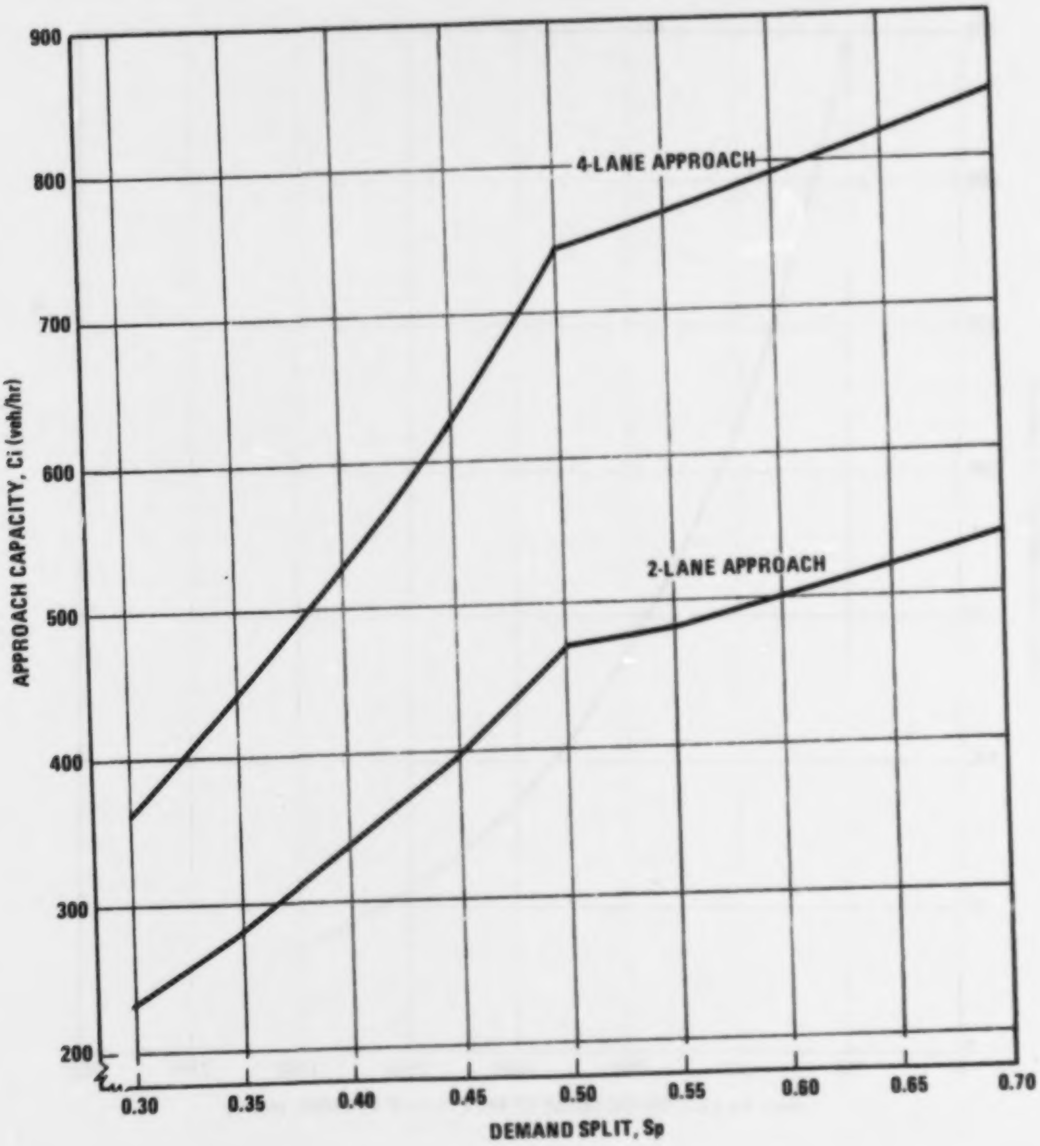

Figure B-3. Capacity at a four-way stop.

(1965) and in Hebert's (1963) study, determines approach capacity. The nomograph is entered with Sp, the ratio of the maximum of the approach volumes on the subject street to the sum of the maximum approach volumes among the street and cross street. (When a three-way-stop T intersection is involved, the one-way volume on the terminal street should be used as the maximum approach volume. Note that the capacity solution presented here will predict capacity such that the maximum V/C on the subject street is equal to the maximum V/C on the cross street.)

5. Gate Capacity

Gate capacities for indirect sources can be estimated for gate entrances and gate exits. The capacity of each exit from a facility is the capacity of the intersection approach leading out of the facility.

The entrance capacity of a parking lot gate is the sum of the through, right-, and left-turning capacity of the approaches that access the gate or road segment. The three contributions to entrance capacity are proportional to the volumes turning onto the road segment. Figure B-4 depicts the volumes turning onto road segment, i, from each connected road. The proportion of vehicles which go through, right, or left are symbolically represented by Pt, Pr, and Pl, respectively. The capacities of approaches i1, i2, and i3, which have access to road segment, i, are represented by C_{i1}, C_{i2}, and C_{i3}. Then the contribution to capacity from each approach which accesses road segment, i, is given by the following:

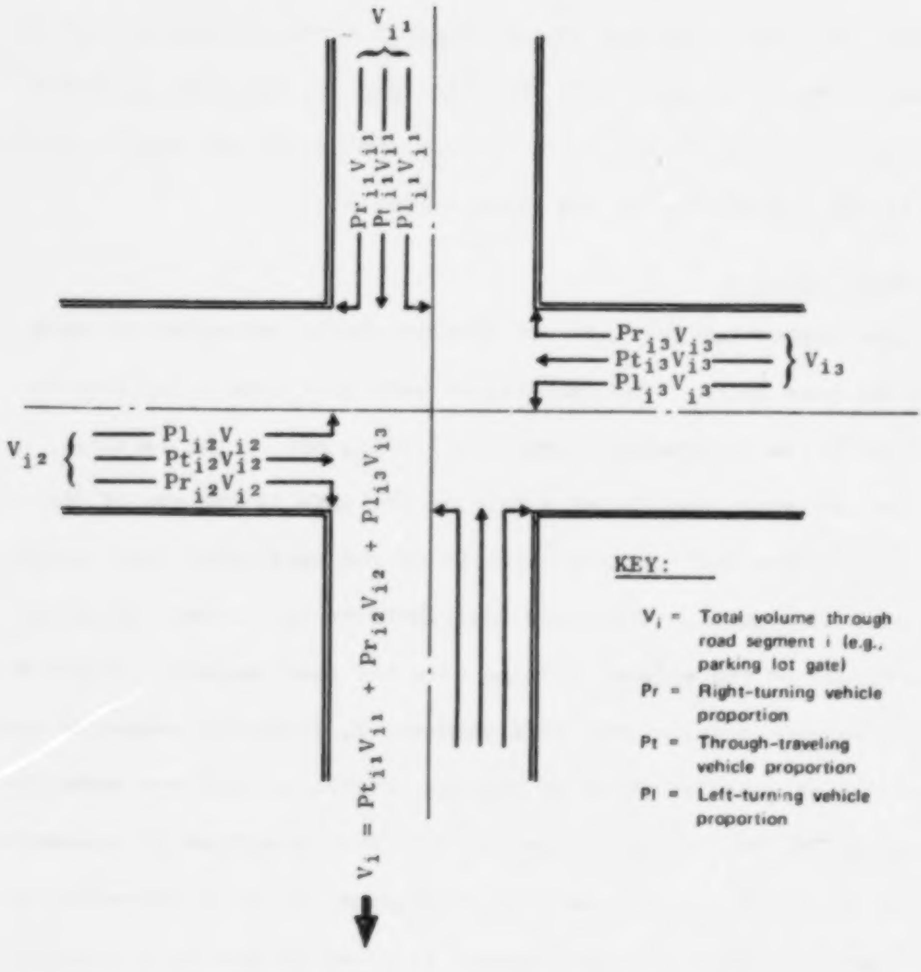
V_i1
Pr_i1 V_i1
Pt_i1 V_i1
Pl_i1 V_i1
Pr_i3 V_i3
Pt_i3 V_i3
Pl_i3 V_i3
V_i3
V_i2
Pl_i2 V_i2
Pt_i2 V_i2
Pr_i2 V_i2
V_i = Pt_i1 V_i1 + Pr_i2 V_i2 + Pl_i3 V_i3
KEY:
V_i = Total volume through road segment i (e.g., parking lot gate)
Pr = Right-turning vehicle proportion
Pt = Through-traveling vehicle proportion
Pl = Left-turning vehicle proportion

FIGURE B-4. SCHEMATIC ILLUSTRATION OF THE APPROACHES AND TURNING MOVEMENTS THAT CONTRIBUTE TO VOLUME (V_i) ON A ROAD SEGMENT

$$Ce_i = \frac{(Pt_{i1})}{100} C_{i1} + \frac{(Pr_{i2})}{100} C_{i2} + \frac{(Pl_{i3})}{100} C_{i3}$$

where

Ce_i is the entrance capacity of road segment i

C_{i1} is the capacity of approach i1 which through traffic accesses the facility

C_{i2} is the capacity of approach i2 from which right-turning vehicles access the facility

C_{i3} is the capacity of approach i3 from which left-turning vehicles access the facility

Pt_{i1} is percentage of vehicles on approach i1 that travel through to road segment i

Pr_{i2} is percentage of right-turning vehicles on approach i2 that turn onto road segment i

Pl_{i3} is percentage of left-turning vehicles on approach i3 that turn onto road segment i.

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Appendix C

STREET CANYON DISPERSION MODEL

Evidence of a helical air circulation in street canyons, as illustrated in Figure C-1, has been observed (see Johnson et al., 1971). This vortex motion forms when the roadway wind angle $\theta \geq 30°$ (See Figure C-2), and the depth of penetration of the rooftop wind into the street canyon, δ, is less than the average height of the upwind buildings, H. The penetration depth, δ, is given by:

$$\delta = 7\ (kW/U)^{1/2} \tag{C-1}$$

where

7 = empirical factor (Georgii, et. al., 1967)

K = turbulent diffusivity of momentum $\sim$ 1 m^2/sec^5

W = street canyon width, m

U = rooftop wind speed, m/sec

Receptors on the leeward side of a building (to the right side as shown in Figures C-1 and C-2) are exposed to substantially higher concentrations than are those on the windward (left) side because of the reverse flow component across the street, near the surface. Thus, we consider the concentration (χ) as a receptor to have two superimposed components. One component is the concentration (χ_b) of the air entering the street canyon from above. The other component ($\Delta\chi$) arises from the locally generated CO emissions within the street. Hence, we have

$$\chi = \chi_b + \Delta\chi \tag{C-2}$$

Equations for calculating the $\Delta\chi$ components on both the leeward side ($\Delta\chi_L$) and the windward side ($\Delta\chi_W$) were derived by Johnson et al. (1971)

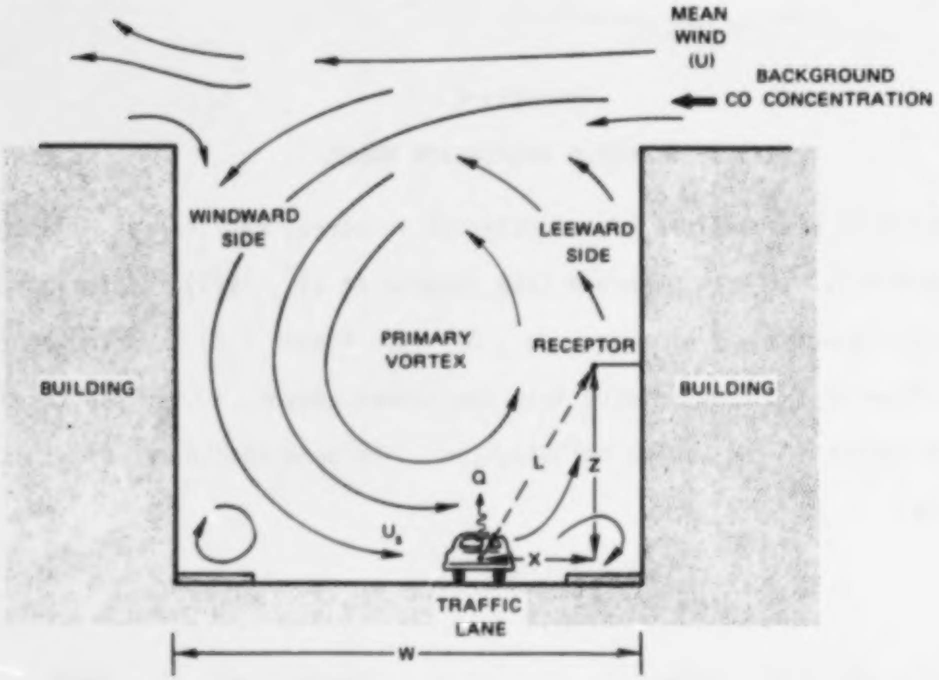
MEAN
WIND
(U)
BACKGROUND
CO CONCENTRATION
WINDWARD
SIDE
LEEWARD
SIDE
PRIMARY
VORTEX
RECEPTOR
BUILDING
BUILDING
L
Z
Q
U_s
X
TRAFFIC
LANE
W

FIGURE C-1 SCHEMATIC OF CROSS-STREET CIRCULATION BETWEEN BUILDINGS

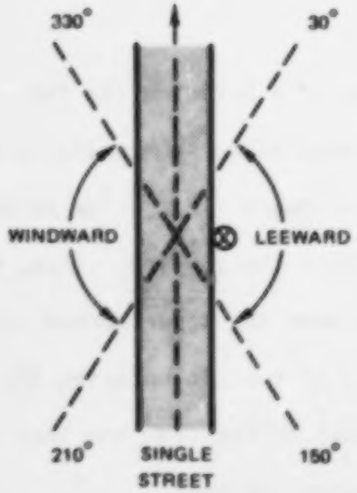
330°
30°
WINDWARD
LEEWARD
210°
150°
SINGLE
STREET

FIGURE C-2 SPECIFICATION FOR LEEWARD AND WINDWARD CASES ON THE BASIS OF RECEPTOR LOCATION, STREET ORIENTATION, AND WIND DIRECTION and modified by Ludwig and Dabberdt (1972). The leeward component is calculated by

$$\Delta x_L = \frac{K\, Q'_s\, (10^3)}{(u+0.5)\ \ (x^2 + z^2)^{1/2} + L_o} \qquad \text{(C-3)}$$

where

$K \simeq 7$, an empirical nondimensional constant

$L_o \simeq$ 2m, approximate vehicle length

x (m), horizontal distance from stream center to receptor

z (m), vertical distance from ground level

u (m/sec), rooftop wind speed, estimate from nearby airport wind speed if local data not available

Q'_s (g/msec), average CO emission rate in the street

x (mg/m^3), CO concentration

The windward-side component (Δx_W) is calculated by

$$\Delta x_W = \frac{K\, Q'_s\, (Hb-z)(10^3)}{W(u+0.5)Hb} \qquad \text{(C-4)}$$

where

W(m), street canyon width

Hb(m), average building height

When the wind direction is such that neither a leeward nor a windward case is appropriate (i.e. $\theta < 30°$ but $\delta <$ Hb) the procedures on the original worksheets should be used.

The above equations apply primarily to street canyons at uninterrupted flow situations, e.g., expressways or midblock locations on arterials. To apply the street canyon calculations at an intersection, care should be exercised in interpreting results. Few studies have thoroughly investigated wind and dispersion patterns at or near intersections. Thus guidance must be given here consistent with the street canyon modeling assumptions, viz., the receptor should be well within the street canyon vortex and not close to the intersection itself. The distance of the receptor into the street canyon should probably be greater than 30m from the cross street curb at the intersection.

In the calculations on the following worksheet only those emissions in the street canyon (i.e., the approach queue, the approach cruise, and the departure cruise emissions from the opposite approach) should be considered. This in effect isolates the street canyon road link from the rest of the intersection but still includes the effects of the intersection on the link in the street canyon, i.e., queueing. In order to be conservative in the CO concentration estimate, this street canyon estimate should be compared to the concentration calculated had no street canyon influence been present, that is, the normal Worksheet 5 calculation. The higher of the two estimates should be used as the representative CO concentration estimate.

After completing Worksheet C, return to the worksheets in main text. Background concentration estimates should be added according to the techniques outlined in Chapter III, Section E.

Instructions for Street Canyon[*]

C-1 Enter Hb (average building height, m) ______

Enter W (average street canyon width, m) ______

C-2 Enter U (rooftop wind speed, estimated or measured, m/sec) ______

C-3 Enter roadway/wind angle (θ) ______

C-4 Calculate δ using equation C-1(m) ______

C-5 Circle one:

$\theta \geq 30°$ and $\delta < Hb$	Street canyon effects possible, continue with Step C-6
$\theta < 30°$ and $\delta < Hb$	Vortex not obvious, return to original worksheet
$\theta \geq 30°$ and $\delta \geq Hb$ $\theta \leq 30°$ and $\delta \geq Hb$	No street canyon effect go back to original worksheet

C-6 Determine if receptor is on windward or leeward side of street (See Figure C-1). Circle one:

windward leeward

C-7 Use the proper equation (leeward, C-3, windward, C-4) to determine the CO concentration. Receptor distances x and z are the same as originally set up by the user. Q_s' is calculated on Worksheet 2.

For free flow $Q'_s = Q_f$ (Worksheet 2, line 18) = ______ (g/sec m)

For intersections $Q'_s = Q_f + Q_e$ = ______ (g/sec m)

where Q_f = Worksheet 2, line 18 for link under consideration plus for link opposite that under consideration

Q_e = Worksheet 2, line 17 for link under consideration

χ = ______ (mg/m^3)

---

[*] At an intersection the instructions apply to only the link where the receptor is located.

References

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Appendix D

A SIMPLE DISPERSION MODEL

1. Model Description

The model to be used here gives the average normalized concentration, $\overline{X}/\overline{Q}$, (i.e., the concentration, χ, averaged over a city and normalized for uniform average area emission rate, Q, as a function of mixing height, H, wind speed, U, and along-wind distance, S', across the city). All units are in meters, seconds, and grams except where indicated otherwise. The main assumptions are:

1. Steady-state conditions prevail.
2. Emissions occur at ground level and are uniform over the city.
3. Pollutants are nonreactive.
4. Lateral diffusion can be neglected.
5. Vertical diffusion from each elemental source conforms to unstable conditions and concentrations follow a Gaussian distribution out to a defined travel time that is a function of H. Thereafter, a uniform vertical distribution of pollutant occurs as a result of further dispersion within the mixing layer.

The model treats the city source as a continuous series of infinitely long cross-wind line sources, much as Lucas (1958) did, with pollutants confined within the mixing layer. As indicated in assumption 5, the model requires two equations according to whether none or some of the pollutants emitted at ground level achieve a uniform vertical distribution within the mixing layer before being transported beyond the downwind edge of the city. These equations are

$$\overline{X}/\overline{Q} = 3.994(S'/U)^{0.115} \qquad \text{(D-1)}$$

for $(S'/U) \leq 0.471H^{1.130}$ (i.e., when no pollutants achieve a uniform vertical distribution), and

---

*Extracted from: Holzworth, G. C., 1972: Mixing heights, wind speeds, and potential for urban air pollution throughout the contiguous United States, EPA Report AP-101, Research Triangle Park, N.C. 27711.

$$\overline{X}/\overline{Q} = 3.613H^{0.130} + \frac{S'}{2HU} - \frac{0.088UH^{1.260}}{S} \qquad \text{(D-2)}$$

for $(S'/U) \geq 0.471H^{1.130}$ (i.e., when some pollutants achieve a uniform vertical distribution). For most cases the term with coefficient 0.088 is very small and can be neglected.

Table D-1 presents the values of $\overline{X}/\overline{Q}$ as a function of H, U, and S'. The variation of $\overline{X}/\overline{Q}$ with S' is practically linear for cities larger than 10 km. Figure D-1 illustrates the variation of (X/Q) over a wider range of city sizes.

In Table D-1, the dashed line separates $\overline{X}/\overline{Q}$ values to the lower right for which H has absolutely no effect for a 10-km city (i.e., all pollutants emitted over a 10-km city are transported beyond the downwind edge of the city before any uniform vertical distribution is achieved within the mixing layer; Eq.D-1 is used). Actually for a given wind speed, $\overline{X}/\overline{Q}$ is practically constant (whole number accuracy) for mixing heights somewhat lower than those for which there is absolutely no effect. This happens because only a small portion of all emissions (i.e., those from near the upwind edge of the city) are affected by the mixing layer before passing beyond the city. In Table D-1, this effect can also be seen for a 100-km city, even though Eq. D-1 is not applicable for a 100-km city for the largest mixing height and wind speed values considered.

An interesting feature of the model is that the larger the city size, the larger the effect an incremental change in U or H has on $\overline{X}/\overline{Q}$ (see Table D-1). This effect is especially large at comparatively small values of U and H, and clearly illustrates the importance of representative data in describing the meteorological potential for air pollution during critical situations. It also indicates that for daily forecasting purposes the input data must be very precise if forecasts are to be reasonably accurate.

Another noteworthy characteristic of the model is that the smaller the values of H and U, and the larger the value of S', the smaller the

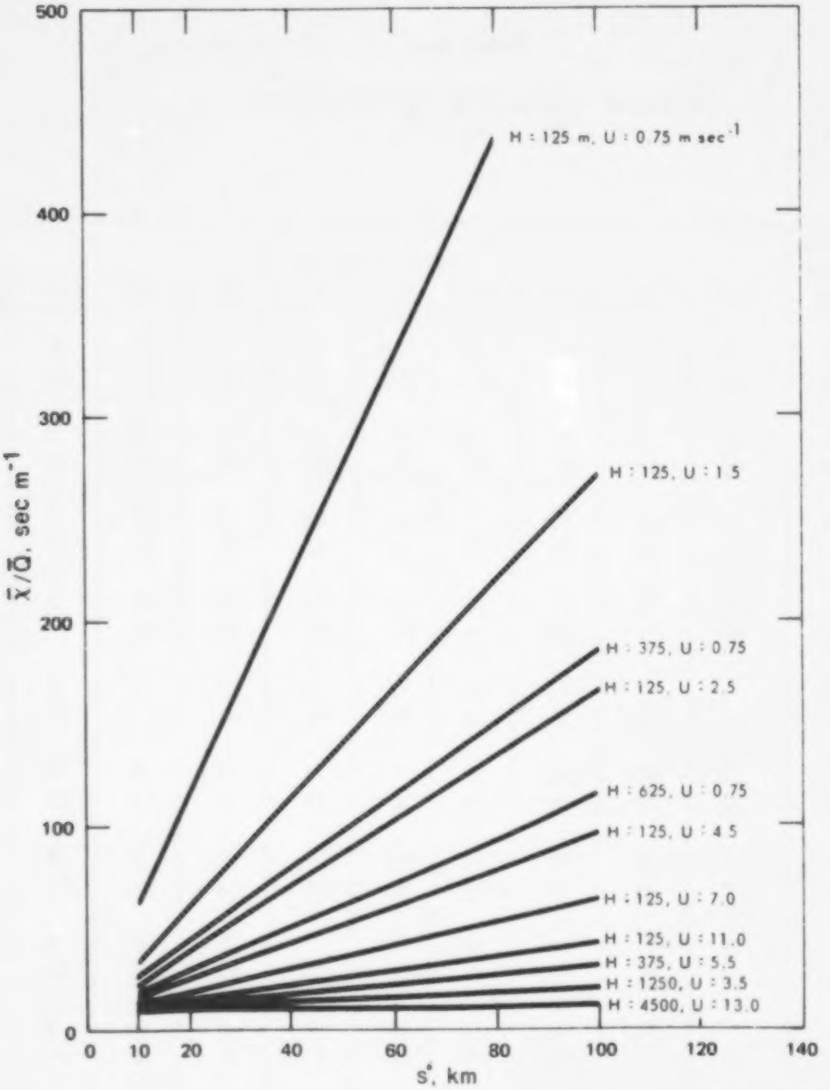

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FIGURE D-1 VARIATION OF χ/Q WITH CITY SIZE (S') FOR VARIOUS COMBINATIONS OF MIXING HEIGHT (H) AND WIND SPEED

Table D-1

AVERAGE NORMALIZED CONCENTRATION
$\overline{\chi}/\overline{Q}$ (sec m^{-1})

City size (km)	Mixing height (m)	Wind speed ($m\ sec^{-1}$)										
		0.75	1.5	2.5	3.5	4.5	5.5	7.0	9.0	11.0	13.0	
10	125	60	33	23	18	16	14	12	11	10	10	
100	125	540	273	167	121	96	79	64	51	43	38	a
10	375	26	17	13	12	11	10	10	9	9	9	
100	375	186	97	61	46	37	32	27	23	20	18	
10	625	19	14	11	11	10	10	9	9	9	9	
100	625	115	62	40	31	26	23	20	17	16	14	
10	875	16	12	11	10	10	10	9	9	9	9	
100	875	85	47	32	25	21	19	17	15	14	13	
10	1250	14	12	11	10	10	9	9	9	9	9	
100	1250	62	36	25	21	18	16	15	14	13	12	
10	1750	13	11	10	10	10	9	9	9	9	9	
100	1750	48	29	21	18	16	15	14	13	12	12	
10	2250	13	11	10	10	10	9	9	9	9	9	
100	2250	39	25	19	16	15	14	13	12	12	11	
10	2750	12	11	10	10	10	9	9	9	9	9	
100	2750	34	22	17	15	14	13	13	12	12	11	
10	3250	12	11	10	10	10	9	9	9	9	9	
100	3250	31	21	16	15	14	13	12	12	11	11	
10	3750	12	11	10	10	10	9	9	9	9	9	
100	3750	28	19	16	14	13	13	12	12	11	11	
10	4500	12	11	10	10	10	9	9	9	9	9	
100	4500	26	18	15	14	13	13	12	12	11	11	

a: dashed line explained in text, page D-2.

relative difference between $\overline{X}/\overline{Q}$ values for this model and those for a "box" model where $(\overline{X}/\overline{Q})_{Box} = 1/2(S'/HU)$. Thus, for $H = 125$ m, $U = 0.75$ m sec^{-1} and $S' = 100$ km, $\overline{X}/\overline{Q} = 540$ sec m^{-1} (Table D-1) and $(\overline{X}/\overline{Q})_{Box} = 533$ sec m^{-1}. This correspondence does not hold, however, for more common values of H, U, and S.

Although the model presented here is rather simple in comparison to the great complexities of atmospheric dispersion and pollutant emissions in urban areas, it is in concert with the general nature of the independent parameters and the spacing of the locations for which mixing height and wind speed are available. As such, it provides a means of quantitatively appraising the general meteorological potential for community air pollution. Obviously, the results of this study will be enhanced by more detailed studies of each local situation.

This model is essentially the same as that for which Miller and Holzworth (1967) obtained good correspondence between calculated and observed concentrations for each of several cities.

2. Concepts and Computation Methods (Mixing Heights, Winds)

The mixing height (or depth) is defined as the height above the surface through which relatively vigorous vertical mixing occurs. The concept of a mixing layer in which the lapse rate is roughly dry adiabatic (unsaturated conditions) is well founded on general theoretical principles and on practical grounds through operational use over several years in the National Air Pollution Potential Forecasting Program (Stackpole, 1967; Gross, 1970). Commonly, mixing heights go through a large diurnal variation. Although not measured directly, they can be calculated approximately from routine meteorological measurements. This study centers on two times of the day, morning and afternoon. The morning mixing height is calculated as the height above ground at which the dry adiabatic extension of the morning minimum surface temperature plus 5°C intersects the vertical temperature profile observed at 1200 Greenwich Mean Time (GMT). The minimum temperature is determined from the regular hourly airways reports from 0200 through 0600 Local Standard Time (LST). The "plus 5°C" is intended to allow roughly for the usual effects of the nocturnal and early morning urban heat island since NWS upper-air-measuring stations are located in rural or suburban surroundings. Thus, more properly, the urban morning mixing height was calculated. The general notion of an urban nocturnal and morning mixing layer, which in reality is often highly complex, is now fairly well established by the investigations of Duckworth and Sandberg (1954). DeMarrais (1961), Summers (1967), and Clark (1969). The value of 5°C was determined arbitrarily after inspection of urban-rural differences in minimum temperature for many locations. The individual differences varied over a larger range and undoubtedly depended upon a number of factors. For general application, however, 5°C is considered a slight over-estimate of an overall average minimum temperature difference--even for existing large cities. For purposes of this report the plus 5°C is interpreted to include the effects of some surface heating shortly after sunrise. Thus, the time of the urban morning mixing height coincides approximately with that of the typical diurnal maximum concentration of slow-reacting pollutants in many cities, occurring around the morning commuter rush hours. This treatment of the urban morning mixing height undoubtedly is a gross simplification of the real situation, but it is considered reasonable for the climatological purposes of this study.

The afternoon mixing height is less complicated than the morning, but was calculated in the same way, except that instead of the minimum temperature plus 5°C, the maximum surface temperature observed from 1200 through 1600 LST was used. Urban-rural differences of maximum surface temperature were assumed negligible. The typical time of the afternoon mixing height may be considered to coincide approximately with the usual mid-afternoon minimum concentration of slow-reacting urban pollutants.

The method described for determining the height of the afternoon mixing (or boundary) layer has been compared with other methods by Hanna (1969), who found it to be the more practical. In addition, mixing heights based on accelerometer and temperature measurements made with a light aircraft during daytime have been found by McCaldin and Sholtes (1970) to be in good agreement with heights calculated as indicated herein (except that McCaldin and Sholtes' calculated heights also made allowance for temperature advection aloft).

Wind speeds for both morning and afternoon were computed as arithmetic averages of speeds observed at the surface and aloft within the mixing layer. Speeds aloft were available for 150 and 300 meters (m) above station elevation and for 500, 1000, 1500, 2000, 3000, 4000 m etc., above sea level. To prevent wind speeds near the same level from being used twice (e.g., as for a station at 190 m above sea level), only winds separated by at least 150 m were used. Morning wind-speed calculations were based on speeds observed aloft at 1200 GMT and an average of the surface speeds observed (regular hourly airways) from 0200 through 0600 LST. Afternoon average speeds were based on the speeds observed aloft at 0000 GMT and the average surface speed from 1200 through 1600 LST. In this report the vertically averaged wind speeds are referred to simply as wind speeds when there is no ambiguity.

In the mixing-height calculations, especially for afternoons, it was assumed implicitly that between the time of a temperature-aloft measurement and a computation time significant changes in vertical temperature structure arose only from heat input at the surface. Certainly, this is not generally true on a day-to-day basis. It is reasonable to assume that over a period of years other influences average out (e.g., that cold air advection is balanced by warm advection). The matter of marked cold air advection, however, did present a problem. For example, when the maximum surface temperature between 1200 and 1600 LST was colder than the surface temperature of the 1200 GMT sounding, the mixing height could not be calculated in the prescribed manner. Such cases were designated type C.

The occurrence of precipitation also demanded special treatment since in such situations the assumption of a dry adiabatic lapse rate in the mixing layer is questionable. Mixing heights (and wind speeds) during significant precipitation were classified as type P. Significant precipitation was defined as at least two occurrences of light or one of moderate or heavy in the regulary hourly airway reports from 1000 through 2100 LST for afternoons and from 2200 through 0900 LST for mornings.

Morning and afternoon mixing heights and wind speeds for 62 stations were calculated and tabulated by the National Climatic Center (NCC), Environmental Data Service (EDS), of the National Oceanic and Atmospheric Administration (NOAA). Most surface and upper-air observations were made from the same location and most calculations were for the five years, 1960 through 1964. The calculations were restricted to five years for economy and to pre-1965 because the required hourly surface observations were on punched cards only through 1964. For most stations, all hourly surface observations through 1964 are readily available in published from (U.S. Department of Commerce) which may be useful in further and/or more detailed studies involving the tabulations. All of the tabulations, which are in three parts for each station, are too lengthy to publish here, but copies may be obtained at the cost of reproduction from the Director, NCC, EDS, NOAA, Asheville, North Carolina 28801.

3. Tabulation of P-, C-, and M-Type Mixing Heights and Wind Speeds

National Climatic Center tabulations of mean mixing heights and wind speeds are given separately for precipitation (P) and non-precipitation (non-P) cases. These tabulations show a distinct tendency for P mixing heights to be higher in the morning and lower in the afternoon than non-P heights. In the calculations, this happens because of the effects of dense cloudiness. Actually, morning and afternoon mixing heights with precipitation may be expected to be higher than without because in the mixing layer above the condensation level the (slower) pseudoadiabatic lapse rate would be more appropriate than the dry abiabatic lapse rate. However, the effectiveness of this consideration is highly dependent on such assumptions as the water vapor content of the initially lifted parcel, the amount of entrainment as the parcel rises, etc. In view of such complexities and the intended climatological use of the derived data, it was decided to allow for all mixing-height and wind-speed cases other than non-P in an arbitrary manner. C cases were treated as P cases since marked cold air advection was assumed to be generally indicative of a comparatively deep mixing layer. Wind speeds for P and C cases were assumed faster than otherwise. The number of missing (M) cases was insignificant.

In allowing for P, C, and M cases, it was assumed that the morning and afternoon mixing heights and wind speeds generally were greater than for non-P cases. The allowance was made through use of frequencies of mixing-height classes by wind speed classes. One-half of the total P, C, and M frequencies were proportionately redistributed among the non-P frequencies for mixing-height classes above the mean height (for all speed classes). The remaining one-half of P, C, and M frequencies were redistributed among the non-P frequencies for wind speed classes above the mean speed (for all mixing height classes). Thus, the non-P part of each table of mixing-height class by wind-speed class was divided into four sections according to the mean height and mean speed. Approximately one-fourth of the P, C, and M frequencies was redistributed in the upper-right section of the frequency table (i.e., in the non-P section with speeds above the mean and heights below the mean); one-fourth was redistributed in the lower left section (i.e., non-P heights above the mean and speeds below); and one-half was redistributed in the lower-right section (i.e., non-P heights and speeds both above the mean). In the redistributions each individual (cell) frequency of non-P mixing height by wind speed was increased in proportion of its frequency to the total non-P frequency of all cells being considered. The total frequencies of all non-P cells above the mean mixing height and above the mean wind speed each was considered separately. Cells with zero non-P frequencies were unaffected by redistribitions as were cells below both the mean mixing height and mean wind speed. Due allowance was made for mean heights and wind speeds that fell within a class interval.

Mean mixing heights and wind speeds are based on averages of the actual values. The means finally arrived at after the redistributions are the NCC Tabulation III means plus the increase in mean value between the mean based on frequency counts by class intervals before (non-P cases only) and after (all cases) the redistributions. Table D-2 gives mean seasonal and annual values of mixing height and wind speed for both before and after allowance for P, C, and M cases. Percentage frequencies of non-P cases are given also.

TABLE D-2
MEAN SEASONAL AND ANNUAL MORNING AND AFTERNOON MIXING HEIGHTS (H) AND WIND SPEEDS (U) FOR NOP[a] AND ALL[b] CASES

Station	Time[c]	Winter					Spring					Summer					Autumn					Annual				
		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}	
		NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All
Albany,	M	576	804	58.0	5.0	6.7	613	786	66.3	5.0	6.1	474	527	83.0	3.7	4.1	491	583	81.1	4.7	4.8	538	675	72.1	4.5	5.4
New York	A	868	967	60.8	6.9	7.6	1663	1753	89.4	8.3	8.7	1896	1943	79.1	6.8	7.0	1231	1272	80.2	6.9	7.2	1414	1484	72.3	7.2	7.6
Albuquerque,	M	345	391	88.9	3.7	4.0	498	553	92.4	4.2	4.5	580	582	93.7	3.6	3.7	377	414	90.3	3.3	3.5	445	485	91.3	3.7	3.9
New Mexico	A	1402	1464	88.3	5.5	5.8	3426	3452	94.4	8.8	8.9	3802	3841	88.5	5.8	6.0	2247	2296	90.1	5.3	5.5	2744	2788	90.3	6.4	6.5
Amarillo,	M	237	273	83.9	6.5	6.9	311	337	88.7	7.8	8.1	353	379	83.5	7.0	7.4	296	323	87.0	6.5	6.8	299	328	85.7	7.0	7.3
Texas		1101	1171	83.0	8.2	8.5	2447	2507	89.6	9.9	10.1	2475	2520	88.9	7.3	7.4	1648	1693	89.4	7.4	7.6	1917	1973	87.4	8.2	8.4
Athens,	M	328	407	71.2	5.1	6.0	328	383	78.7	4.7	5.3	363	390	82.6	3.6	3.8	278	314	85.5	4.0	4.4	324	374	79.5	4.4	4.9
Georgia	A	970	1042	72.4	6.5	7.0	1707	1754	77.8	6.9	7.2	1876	1918	75.7	4.7	4.9	1428	1455	85.3	5.5	5.7	1495	1547	77.8	5.9	6.2
Bismarck,	M	272	380	59.7	5.0	6.5	378	474	68.7	5.5	6.5	239	282	80.2	3.9	4.5	255	301	84.4	4.4	5.0	286	359	73.2	4.7	5.6
North Dakota	A	528	625	60.8	7.0	7.8	1756	1880	74.6	8.7	9.2	2015	2078	84.4	7.0	7.2	1299	1368	83.5	7.8	8.1	1399	1488	75.8	7.6	8.1
Boise,	M	327	407	68.1	3.6	4.2	342	424	76.7	4.5	5.0	185	193	90.4	3.3	3.4	224	279	79.8	3.8	4.2	269	326	78.7	3.8	4.2
Idaho	A	631	754	67.9	4.3	4.9	2244	2329	78.0	6.4	6.7	2511	2540	92.2	5.8	5.9	1320	1409	81.5	5.0	5.3	1646	1758	79.9	5.4	5.7
Brownsville	M	386	438	74.6	6.4	7.4	722	746	88.9	8.1	8.3	788	794	96.5	7.1	7.2	530	561	89.2	5.6	5.9	601	635	87.3	6.8	7.2
Texas	A	1043	1084	78.3	8.2	8.6	1113	1127	89.4	9.5	9.6	1533	1540	93.0	8.7	8.7	1369	1385	86.8	7.7	7.8	1264	1284	86.8	8.5	8.7
Buffalo,	M	595	869	36.6	6.3	8.2	476	627	62.5	5.4	6.4	391	458	80.7	4.2	4.6	462	571	76.7	4.7	5.3	481	631	64.1	5.1	6.1
New York	A	727	857	34.4	7.2	8.5	1333	1431	56.8	7.5	8.1	1575	1616	81.0	6.5	6.7	1135	1196	73.1	6.5	6.9	1192	1275	61.3	6.9	7.6
Burrwood,	M	536	593	81.6	7.0	7.5	701	720	92.6	6.4	6.6	1282	1300	87.8	4.1	4.2	1090	1113	87.5	6.2	6.4	902	932	87.3	5.9	6.2
Louisiana	A	637	681	76.1	6.9	7.1	804	816	91.3	6.1	6.2	1238	1252	83.7	4.2	4.3	1053	1073	83.1	6.0	6.2	933	956	83.5	5.8	5.9
Cape Hatteras,	M	630	725	73.2	7.8	8.5	643	736	78.3	7.6	8.2	780	848	77.4	6.2	6.7	761	846	78.2	6.6	7.2	703	789	76.7	7.0	7.6
North Carolina	A	701	769	71.5	7.7	8.2	878	945	77.2	8.5	8.8	988	1021	78.7	7.1	7.3	923	962	78.7	7.0	7.3	872	924	76.5	7.6	7.8
Caribou,	M	447	586	54.0	6.1	7.5	475	610	62.8	5.6	6.7	417	508	68.7	4.4	5.1	477	603	65.5	5.3	6.3	452	579	62.7	5.4	6.4
Maine	A	706	823	49.6	7.8	8.9	1511	1618	62.0	7.7	8.7	1817	1883	69.8	7.6	8.0	1099	1191	62.4	7.7	8.3	1283	1379	60.9	7.7	8.3
Charleston,	M	296	363	74.8	4.9	5.8	339	385	82.6	4.8	5.3	411	437	84.4	4.2	4.5	264	297	85.7	4.2	4.6	327	371	81.8	4.5	5.1
South Carolina	A	951	1004	75.0	6.7	7.1	1519	1562	82.4	7.2	7.4	1447	1510	70.7	5.7	6.0	1222	1243	84.8	6.1	6.3	1204	1330	78.2	6.4	6.7
Columbia,	M	390	448	74.3	6.0	6.5	409	477	73.9	6.6	7.3	294	321	83.3	4.7	5.0	317	358	83.1	5.5	5.9	357	401	78.6	5.7	6.2
Missouri	A	797	872	71.5	7.0	7.5	1523	1599	75.8	8.4	8.8	1689	1723	83.9	5.6	5.8	1349	1395	82.7	6.5	6.7	1339	1397	78.3	6.9	7.2
Dayton,	M	461	557	58.6	6.7	7.1	462	555	67.2	5.9	6.6	349	375	87.4	3.8	4.0	360	417	82.4	4.5	5.0	406	476	73.9	5.1	5.7
Ohio	A	789	836	56.4	7.2	7.9	1570	1670	68.3	7.6	8.0	1661	1685	83.9	5.3	5.5	1315	1346	82.6	6.3	6.5	1373	1384	72.8	6.6	7.0

[a] NOP excludes type P, C, and M cases (see text).

[b] ALL includes all cases, with allowances for types P, C, and M (see text).

[c] M, morning; A, afternoon.

TABLE D-2 (Continued)
MEAN SEASONAL AND ANNUAL MORNING AND AFTERNOON MIXING HEIGHTS (H) AND WIND SPEEDS (U) FOR NOP^a AND ALL^b CASES

Station	Time^c	Winter					Spring					Summer					Autumn					Annual				
		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}	
		NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All
Denver, Colorado	M	178	219	78.5	4.5	4.8	380	423	82.6	4.3	4.6	243	255	92.6	3.6	3.7	163	174	87.0	3.5	3.6	236	268	85.1	4.0	4.2
	A	1357	1482	80.3	5.8	6.3	2951	3070	80.7	7.2	7.6	3258	3458	79.6	5.9	6.1	2086	2161	96.8	5.1	5.3	2437	2543	81.8	6.0	6.3
Dodge City, Kansas	M	224	266	80.5	6.7	7.3	316	354	81.7	7.7	8.2	302	328	79.6	7.1	7.5	256	283	85.5	6.6	7.0	274	308	81.8	7.0	7.5
	A	811	879	81.4	8.0	8.3	1783	1872	82.8	9.6	9.9	2028	2086	86.1	8.3	8.5	1267	1323	87.3	8.2	8.4	1472	1540	84.4	8.5	8.8
El Paso, Texas	M	360	405	90.7	4.2	4.5	660	680	96.1	5.5	5.7	718	758	89.8	4.0	4.2	386	429	90.3	3.3	3.5	530	571	91.7	4.3	4.5
	A	1421	1460	91.6	5.9	6.1	3211	3222	97.0	8.4	8.4	3696	3721	91.7	5.1	5.2	2118	2148	91.9	4.4	4.5	2611	2638	93.0	5.9	6.1
Ely, Nevada	M	167	193	78.8	4.8	5.1	427	469	83.3	4.7	5.1	108	109	95.0	4.2	4.2	146	161	88.8	4.3	4.5	209	230	86.4	4.5	4.7
	A	1030	1072	80.8	5.2	5.5	2630	2708	79.1	7.1	7.4	3583	3624	87.8	6.9	7.0	2116	2179	88.1	5.9	6.1	2337	2396	83.9	6.3	6.5
Flint, Michigan	M	518	674	49.1	6.0	7.2	439	527	89.6	5.3	6.1	280	328	83.5	3.5	4.0	400	494	77.1	4.5	5.2	406	506	69.8	4.8	5.6
	A	762	862	45.6	7.0	7.8	1466	1609	86.1	7.6	8.2	1687	1734	81.1	6.1	6.3	1213	1268	75.6	6.7	7.1	1292	1368	67.1	6.8	7.4
Glasgow, Montana	M	220	283	73.7	4.7	5.5	347	381	82.6	5.9	6.3	277	304	86.5	5.4	5.7	232	262	88.8	5.0	5.3	260	310	82.9	5.3	5.7
	A	426	524	67.3	6.2	6.9	1866	1971	77.6	7.7	8.1	2409	2494	88.0	7.1	7.2	1257	1307	88.6	7.5	7.7	1487	1564	80.3	7.1	7.5
Grand Junction, Colorado	M	276	329	81.9	3.1	3.4	560	628	86.5	5.1	5.4	280	307	90.4	4.6	4.7	238	273	86.6	3.7	3.9	338	384	86.3	4.1	4.3
	A	1075	1160	83.0	3.1	3.4	3087	3166	86.1	6.3	6.6	3805	3940	67.8	5.9	6.1	2048	2133	86.4	4.4	4.6	2526	2600	85.8	4.9	5.2
Great Falls, Montana	M	447	562	73.5	8.8	9.7	527	643	70.7	6.5	7.3	368	380	80.1	4.4	4.7	422	491	84.2	7.0	7.5	438	524	80.8	6.7	7.3
	A	874	1003	70.8	9.9	10.5	2318	2439	72.2	8.4	8.9	2984	3040	83.3	6.9	7.2	1641	1707	83.3	9.0	9.3	1954	2047	77.4	8.6	9.0
Green Bay, Wisconsin	M	442	573	58.7	5.8	6.9	433	534	70.9	5.4	6.2	323	372	81.7	4.0	4.5	465	543	79.6	4.9	5.5	415	506	72.9	5.0	5.8
	A	632	704	58.7	7.3	7.9	1380	1492	73.3	7.8	8.2	1607	1648	84.1	6.5	6.7	1067	1127	78.0	7.1	7.5	1173	1243	73.7	7.2	7.6
Greensboro, North Carolina	M	380	480	74.8	4.7	5.4	402	492	76.1	4.7	5.4	400	445	80.9	3.8	4.2	290	343	82.4	3.8	4.3	370	440	78.5	4.3	4.8
	A	926	982	71.9	6.4	6.8	1708	1765	78.7	7.1	7.4	1674	1710	78.5	4.9	5.1	1306	1334	84.6	5.6	5.8	1403	1450	78.4	6.0	6.3
Huntington, West Virginia	M	482	634	57.9	4.2	5.3	575	721	70.7	4.6	5.5	300	336	83.0	2.4	2.7	332	403	82.1	2.6	3.1	442	524	73.4	3.4	4.2
	A	963	1079	58.3	5.7	6.4	1672	1986	72.5	6.0	6.5	1596	1641	81.9	4.1	4.3	1300	1340	81.7	4.6	4.9	1432	1511	73.6	5.1	5.5
International Falls, Minn.	M	251	347	54.0	4.3	5.6	319	411	66.3	4.6	5.6	286	337	76.2	3.3	4.1	406	513	70.6	5.1	6.0	310	402	66.5	4.3	5.3
	A	584	696	52.7	6.3	7.0	1540	1646	68.3	7.1	7.5	1688	1747	78.9	6.6	6.9	1054	1146	69.9	7.0	7.4	1216	1299	67.4	6.8	7.2
Jackson, Mississippi	M	370	470	68.8	4.0	4.9	417	467	81.3	4.4	4.9	408	421	91.9	3.0	3.1	395	343	83.0	3.2	3.6	377	425	81.2	3.7	4.1
	A	1014	1088	69.1	5.4	5.9	1503	1543	81.8	6.0	6.2	1803	1830	78.4	4.3	4.5	1310	1349	31.3	4.9	5.1	1409	1453	77.1	5.2	5.4
Jacksonville, Florida	M	345	403	79.4	5.2	5.9	447	477	90.4	5.3	5.6	567	583	91.1	4.3	4.4	418	466	85.9	4.7	5.0	444	480	86.7	4.9	5.2
	A	1058	1104	80.1	6.7	7.0	1639	1667	86.1	7.1	7.2	1681	1712	88.0	5.6	5.8	1321	1342	80.4	6.5	6.5	1424	1456	78.6	6.5	6.7
Lake Charles, Louisiana	M	319	384	71.2	5.8	6.7	418	450	87.3	5.3	5.7	493	506	92.4	4.0	4.1	280	308	90.8	4.3	4.5	380	417	85.4	4.9	5.3
	A	822	867	74.5	7.1	7.5	1164	1175	86.2	6.0	7.0	1365	1392	80.4	5.0	5.2	1299	1315	86.5	5.7	5.8	1162	1186	81.9	6.2	6.4

TABLE D-2 (Continued)
MEAN SEASONAL AND ANNUAL MORNING AND AFTERNOON MIXING HEIGHTS (H) AND WIND SPEEDS (U) FOR NOP^a AND ALL^b CASES

Station	Time^c	Winter					Spring					Summer					Autumn					Annual				
		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}	
		NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All
Lander, Wyoming	M	188	723	83.0	2.5	2.8	427	511	81.7	3.2	3.7	326	337	95.4	2.7	2.7	265	327	86.2	2.6	2.9	301	348	86.5	2.8	3.0
	A	808	926	82.1	3.6	4.1	2629	2755	79.4	6.4	6.9	3405	3480	84.4	6.3	6.6	1901	2030	85.1	4.6	5.0	2187	2300	87.7	5.7	5.6
Las Vegas, Nevada	M	266	321	92.0	4.2	4.5	405	433	95.4	5.4	5.6	283	292	97.2	4.6	4.7	242	276	94.3	4.1	4.3	299	331	94.7	4.6	4.8
	A	1123	1153	94.0	4.1	4.2	2700	2785	95.2	7.0	7.1	3872	3893	92.0	6.6	6.7	2076	2106	95.6	5.1	5.2	2410	2434	94.2	5.7	5.8
Little Rock, Arkansas	M	451	541	76.3	4.6	5.2	460	544	77.8	5.0	5.7	355	375	90.0	3.5	3.7	303	342	86.2	3.4	3.8	392	450	82.5	4.1	4.6
	A	995	1101	72.1	6.1	6.6	1554	1612	80.0	6.7	7.0	1817	1851	84.1	4.8	4.9	1380	1401	84.6	5.0	5.2	1431	1491	80.2	5.7	5.9
Medford, Oregon	M	289	387	60.6	1.5	1.9	392	535	68.0	1.8	2.2	250	285	93.7	1.2	1.3	220	293	75.0	1.1	1.3	290	375	74.3	1.4	1.7
	A	747	933	65.0	2.2	2.8	2004	2079	67.6	4.5	4.8	2332	2349	92.0	4.6	4.6	1481	1504	77.6	3.2	3.5	1641	1738	75.5	3.6	3.9
Miami, Florida	M	654	707	87.2	5.4	5.7	947	980	91.1	5.7	5.9	1041	1071	88.3	4.3	4.5	872	933	82.4	5.0	5.3	878	923	87.2	5.1	5.3
	A	1208	1221	89.2	6.4	6.5	1440	1450	87.4	6.8	6.9	1380	1383	73.7	5.3	5.5	1315	1341	78.7	6.6	6.9	1330	1351	82.2	6.3	6.5
Midland, Texas	M	240	280	83.4	5.1	5.7	405	429	90.9	7.3	7.5	503	606	90.2	7.0	7.2	380	419	87.9	5.7	6.0	406	436	88.1	6.3	6.6
	A	1222	1276	84.5	7.5	7.8	2408	2449	92.4	8.9	9.0	2716	2744	90.7	6.6	6.7	1049	1087	89.7	6.6	6.7	2048	2089	89.3	7.4	7.5
Montgomery, Alabama	M	387	484	72.1	4.2	5.0	382	431	82.6	3.9	4.3	430	444	92.0	3.4	3.5	294	323	86.8	3.3	3.6	373	420	83.3	3.7	4.1
	A	988	1060	72.1	5.8	6.3	1590	1622	84.4	5.8	6.0	1770	1801	79.6	4.3	4.4	1380	1402	85.7	4.8	4.9	1432	1471	80.4	5.2	5.4
Nantucket, Massachusetts	M	780	905	63.7	8.5	9.5	588	734	66.3	7.5	8.4	380	448	75.9	5.7	6.2	625	739	72.3	6.9	7.6	595	707	69.5	7.1	7.9
	A	791	890	59.7	9.0	9.8	746	827	74.6	8.4	8.8	600	667	77.4	7.1	7.3	765	831	70.8	7.9	8.3	727	804	70.6	8.1	8.6
Nashville, Tennessee	M	440	563	62.8	4.3	5.2	500	606	73.7	4.8	5.6	417	441	80.4	3.2	3.4	301	357	81.1	3.0	3.5	414	492	76.7	3.8	4.4
	A	1035	1123	66.6	6.2	6.8	1713	1703	75.4	6.8	7.1	1045	1074	80.9	4.6	4.8	1438	1473	80.9	5.1	5.3	1507	1563	75.9	5.7	6.0
New York, New York	M	875	986	71.5	8.3	9.1	788	918	72.0	6.9	7.7	662	711	82.0	5.5	5.8	675	741	82.6	6.6	7.1	750	839	77.0	6.8	7.4
	A	901	976	67.5	8.2	8.8	1380	1466	75.4	8.7	9.1	1512	1570	82.0	6.8	7.0	1086	1132	80.7	7.4	7.7	1214	1286	76.4	7.8	8.2
North Platte, Nebraska	M	288	294	72.1	4.3	5.3	317	383	73.0	5.9	6.8	321	361	80.9	5.4	5.9	238	287	82.0	4.3	4.9	271	329	77.0	5.0	5.7
	A	886	986	77.4	7.6	8.1	1778	1884	76.1	8.7	9.1	1717	1756	88.7	7.3	7.4	1356	1400	88.6	7.5	7.7	1434	1509	82.7	7.0	8.1
Oakland, California	M	386	453	78.3	2.0	3.3	701	763	84.6	3.9	4.2	515	527	93.3	3.2	3.3	464	508	85.3	2.7	2.9	516	563	86.3	3.2	3.4
	A	649	709	80.1	4.1	4.4	1087	1121	87.6	6.5	6.6	643	644	98.0	5.9	5.9	745	770	89.5	5.0	5.1	781	811	88.8	5.4	5.5
Oklahoma City, Oklahoma	M	296	342	76.1	6.9	7.5	409	457	82.0	8.7	9.2	344	387	83.5	6.9	7.2	300	343	83.1	6.7	7.2	330	377	81.6	7.3	7.8
	A	804	859	78.8	8.1	8.4	1447	1506	82.0	9.8	10.0	1839	1862	88.5	7.2	7.3	1266	1302	85.5	7.7	7.9	1336	1382	83.7	8.2	8.4
Peoria, Illinois	M	327	382	68.3	5.2	5.9	361	431	71.1	5.7	6.4	272	305	83.3	3.8	4.2	273	321	82.2	4.4	4.9	308	382	76.4	4.8	5.4
	A	533	584	64.2	6.8	7.3	1353	1443	72.2	8.2	8.7	1498	1532	85.0	5.8	6.0	1088	1104	83.7	6.7	6.9	1113	1168	76.2	6.9	7.2
Pittsburgh, Pennsylvania	M	419	634	42.3	4.7	6.5	404	536	61.1	4.2	5.1	333	382	82.8	3.1	3.5	404	488	79.3	3.7	4.3	390	510	66.3	3.9	4.9
	A	811	920	45.1	6.9	7.9	1753	1892	60.9	7.7	8.4	1794	1827	82.2	5.4	5.6	1385	1409	78.7	6.2	6.5	1430	1512	66.7	6.5	7.1

TABLE D-2 (Continued)
MEAN SEASONAL AND ANNUAL MORNING AND AFTERNOON MIXING HEIGHTS (H) AND WIND SPEEDS (U) FOR NOP[a] AND ALL[b] CASES

Station		Winter					Spring					Summer					Autumn					Annual				
		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}	
		NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All
Portland, Maine	M	461	618	70.4	5.6	6.6	533	689	68.5	5.4	6.4	435	501	78.9	4.3	4.8	395	506	74.1	4.7	5.6	461	578	72.9	5.0	5.9
	A	870	945	69.0	7.2	7.8	1284	1479	70.9	8.2	8.7	1414	1476	80.4	7.3	7.6	1083	1141	76.3	7.0	7.4	1190	1260	74.1	7.4	7.8
Rapid City, South Dakota	M	276	363	65.5	4.7	6.3	382	465	73.7	5.8	6.7	298	330	85.4	4.5	4.8	264	314	86.6	5.0	5.5	287	368	77.8	5.0	5.8
	A	848	982	67.7	7.2	7.9	2032	2166	74.6	8.4	8.8	2419	2510	83.5	6.7	6.9	1569	1615	87.0	7.6	7.8	1714	1810	78.2	7.5	7.9
St. Cloud, Minnesota	M	338	393	74.8	5.4	6.1	404	469	77.6	5.6	6.3	328	351	89.4	3.9	4.2	389	429	87.0	5.1	5.5	364	411	82.2	5.0	5.5
	A	537	607	59.7	6.6	7.3	1344	1432	75.7	7.7	8.0	1505	1646	82.8	6.6	6.9	952	1086	80.7	7.4	7.7	1107	1173	74.7	7.1	7.5
Salem, Oregon	M	325	431	56.2	3.0	3.8	432	627	55.4	2.3	2.9	379	424	89.1	2.0	2.2	292	404	68.1	2.2	2.8	357	471	67.2	2.4	2.8
	A	622	787	50.4	3.7	4.5	1614	1733	56.3	4.3	4.8	1666	1632	88.1	4.4	4.5	1115	1212	67.9	4.2	4.6	1251	1354	65.1	4.1	4.6
Salt Lake City, Utah	M	294	329	71.9	3.7	4.3	317	419	73.7	4.8	5.4	198	216	88.9	4.4	4.6	198	238	84.8	4.3	4.8	241	300	79.8	4.3	4.7
	A	808	944	67.9	4.0	4.6	2540	2675	74.8	6.2	6.6	3673	3737	85.9	6.0	6.2	1821	1933	83.7	5.2	5.5	2212	2322	78.0	5.3	5.7
San Antonio, Texas	M	370	450	71.7	5.0	5.8	678	748	70.9	6.0	6.5	897	915	92.0	5.8	5.7	585	654	84.0	5.1	3.5	635	694	81.6	5.4	5.9
	A	1065	1112	70.0	6.5	6.8	1529	1552	88.7	7.1	7.2	2108	2119	91.7	5.9	6.0	1542	1572	87.5	6.0	6.1	1561	1580	86.7	6.4	6.5
San Diego, California	M	468	534	88.1	2.0	2.2	807	851	91.1	2.6	2.7	531	538	95.4	2.0	2.0	568	578	95.0	2.0	2.1	501	625	92.4	2.2	2.2
	A	989	1021	91.4	4.2	4.3	1056	1085	92.6	5.1	5.2	564	586	98.5	4.1	4.1	819	834	95.8	4.3	4.3	857	877	94.5	4.4	4.5
Santa Monica, California	M	376	422	85.4	2.8	3.0	651	676	92.6	2.7	2.8	562	582	94.1	1.9	1.9	467	510	91.9	2.3	2.4	516	542	91.0	2.4	2.5
	A	863	893	91.6	4.4	4.5	946	983	95.0	5.9	6.0	603	603	99.4	5.1	5.1	785	798	95.8	4.8	4.8	799	814	95.4	5.1	5.1
Sault Ste. Marie, Michigan	M	386	532	33.9	4.8	6.1	380	493	66.1	4.7	5.5	270	332	76.1	3.7	4.2	519	676	64.2	4.9	5.8	383	508	60.0	4.5	5.4
	A	582	709	33.4	5.9	7.0	1152	1243	66.3	7.3	7.7	1294	1433	79.1	6.9	7.1	1024	1125	60.7	7.0	7.6	1038	1125	59.8	6.8	7.3
Seattle, Washington	M	626	824	49.8	5.1	6.2	681	839	55.1	4.6	5.5	532	576	85.1	4.0	4.2	476	585	61.5	4.3	5.0	570	705	62.8	4.5	5.2
	A	585	718	45.8	4.7	5.4	1480	1577	56.5	5.7	6.2	1398	1419	89.5	4.8	4.9	898	987	56.3	4.6	5.0	1097	1175	64.5	4.9	5.4
Shreveport, Louisiana	M	430	508	72.4	5.7	6.3	5.5	586	83.9	6.3	6.7	489	482	92.2	4.7	4.8	381	430	88.2	4.5	4.8	451	487	83.6	5.3	5.7
	A	1016	1088	72.0	6.2	6.7	1441	1484	82.4	6.8	7.1	1002	1023	85.4	4.7	4.8	1385	1414	85.3	5.2	5.4	1411	1452	81.4	5.7	6.0
Spokane, Washington	M	336	414	60.2	4.7	5.7	341	401	74.1	5.2	5.8	236	259	88.3	4.1	4.4	218	266	75.0	3.8	4.3	282	335	74.4	4.4	5.1
	A	430	523	56.9	4.8	5.6	1861	1943	72.0	6.1	6.5	2533	2569	87.8	5.2	5.3	1261	1382	74.7	5.0	5.4	1521	1597	72.8	5.3	5.7
Tampa, Florida	M	394	436	85.8	5.8	6.1	503	526	91.7	5.6	5.8	656	674	91.1	4.2	4.3	419	439	89.2	5.4	5.6	493	519	89.4	5.3	5.4
	A	1052	1070	81.4	6.4	6.6	1523	1544	87.8	6.7	6.8	1460	1526	88.9	5.0	5.3	1401	1429	84.4	6.4	6.8	1359	1394	80.6	6.2	6.4
Topeka, Kansas	M	381	448	73.7	5.9	6.6	454	529	75.9	7.0	7.8	371	408	80.4	5.2	5.6	317	381	81.3	5.0	5.6	380	438	77.8	5.8	6.4
	A	881	876	74.6	7.4	7.9	1441	1512	77.2	9.3	9.6	1503	1547	86.1	6.8	7.0	1228	1267	82.2	7.5	7.8	1243	1299	80.0	7.8	8.1
Tucson, Arizona	M	216	247	90.7	4.3	4.5	244	260	98.5	4.2	4.3	335	356	90.0	3.8	3.9	275	341	93.0	4.4	4.5	255	276	92.5	4.2	4.3
	A	1380	1424	88.3	5.0	5.7	2669	2664	97.8	6.8	6.8	3040	3110	82.2	5.3	5.5	2073	2110	89.5	4.9	5.0	2290	2327	89.4	5.5	5.6

TABLE D-2 (Concluded)
MEAN SEASONAL AND ANNUAL MORNING AND AFTERNOON MIXING HEIGHTS (H) AND WIND SPEEDS (U) FOR NOP[a] AND ALL[b] CASES

		Winter					Spring					Summer					Autumn					Annual				
		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}		H, m			U, m sec^{-1}	
Station		NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All	NOP	All	% NOP	NOP	All
Washington, D.C.	M	539	672	74.0	5.3	6.3	481	586	77.5	4.7	5.4	378	421	84.0	3.1	3.4	359	436	83.2	3.7	4.4	439	528	79.6	4.2	4.9
	A	983	1054	71.5	6.7	7.3	1795	1886	74.5	7.5	7.9	1884	1924	83.4	5.4	5.6	1371	1412	84.3	6.2	6.4	1503	1570	78.4	6.4	6.8
Winnemucca, Nevada	M	231	301	83.2	2.8	3.3	343	434	83.0	3.6	4.1	117	129	85.2	2.6	2.7	179	255	82.6	2.9	3.4	217	280	83.5	3.0	3.4
	A	1001	1067	82.5	4.5	4.9	2699	2756	83.5	6.6	6.8	3627	3666	92.6	6.1	6.2	2095	[illegible]	90.3	5.2	5.4	2266	2407	87.2	5.6	5.8
Winslow, Arizona	M	205	223	91.1	2.8	3.0	261	279	94.2	4.0	4.2	221	232	93.8	3.2	3.3	198	213	91.9	2.5	2.6	216	234	92.7	3.1	3.3
	A	1078	1128	91.5	5.3	5.6	3160	3178	94.6	8.8	8.8	3801	3840	87.3	6.9	7.1	2243	2303	88.3	5.0	5.2	2570	2613	90.4	6.5	6.7

4. Background Concentrations

Because of the nature of the model presented here, that is, the assumptions presented in Section D-1, the concentration estimates are most appropriate for short-term averaging times. These concentrations are based on a city-wide emission and dispersion average and are thus representative of a 1-hour city-averaged concentration. Because of wind and mixing height fluctuations, however, estimate of an 8-hour background concentration cannot be made directly using the technique previously presented. To obtain an 8-hour estimate a persistence factor similar to those discussed in Section III-E may be multiplied times the 1-hour concentration obtained from the Holzworth model. This 8-hour concentration will be most conservative if the city-wide averaged value is:

1. Computed when city-wide emissions are highest.
2. Local meteorological conditions are poor in terms of dispersion.

These 1-hour and 8-hour concentration estimates may be used in lieu of background monitoring data, when it is unavailable, to estimate total CO impact as described in Section III-E.

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APPENDIX E

HIWAY

The EPA HIWAY Model (Zimmerman and Thompson, 1975) is a short-term Gaussian model providing estimates for averaging times of about one hour. Traffic emissions are simulated by assuming uniform emissions over a straight-line source of finite length for each lane of the highway. To simplify the analysis in these guidelines a stream (consolidation of two or more lanes) is used to simulate the highway under consideration. This allows the use of normally available highway statistics and reduces the number of calculations while not significantly reducing the precision of the concentration estimates (< 2%).

Air pollution concentrations downwind from each line source are determined by a numerical integration along the line source of a simple Gaussian point source plume. Initial spreading of the pollutant in the turbulent wake of vehicle traffic is modeled by specifying appropriate values for the standard deviations of pollutant distributions (i.e., initial dispersion coefficients σ_{y_o}, σ_{z_o}). The HIWAY Model requires information about highway geometry, automotive emissions and meteorological conditions, all of which are input in the indirect source analysis herein.

Because HIWAY assumes steady state conditions it would not be expected to perform well under very low wind speeds, say, less than 1 m/sec. During such light wind conditions, the wind direction frequently meanders over wide ranges, hence, steady state conditions are not closely approximated. HIWAY performs reasonably well for wind-roadway angles perpendicular through parallel, with worst concentration overestimates under parallel, light wind (< 1 m/sec), very stable conditions (F). Hence, concentration estimates should not be made with a road with an angle of less than about 5-8°, or with wind speeds less than 1 m/sec.

The primary reason for using the HIWAY model in this analysis in lieu of say, CALINE-2 or others, is that HIWAY has the needed capability for modeling finite line sources, such as queueing vehicles. Other lesser reasons include that HIWAY has been used widely and generally been accepted as a usable modeling technique for mobile sources.

REFERENCE

Zimmerman, John R. and R. S. Thompson, 1975: "User's Guide for HIWAY, a Highway," EPA Report No. 650/4-74-008, Research Triangle Park, North Carolina 27711.

APPENDIX F

CONGESTED CONDITIONS

"Prediction of traffic flow under congested conditions is perhaps a thousand times more difficult than under non-congested conditions," according to Professor Dolf May, University of California at Berkeley.* The Guidelines themselves indicate that if a freeway or intersection becomes congested due to the construction of an indirect source, then the capacity of the impacted roadways should be expanded. However, such construction may be impractical; also, there is a need to understand the complexity of analysis under congested conditions rather than to deny approval of projects with congested conditions. The technical Guidelines are intended to offer a simple yet comprehensive procedure, but are not intended to give a complex analysis of congestion. The following is a brief discussion of the problem of evaluating vehicle emissions under congested conditions.

Congestion occurs when demand volume exceeds the capacity of a roadway or intersection. The demand volume is the number of vehicles that desire to use a roadway or intersection during a period of time, usually one hour. During congested flow the volume actually using a roadway or intersection is less than the demand volume.

Congestion with regard to freeway flow can be due to the effects of merging, weaving, or too few lanes. The section causing the congestion is called a bottleneck. The effect of a bottleneck is to:

- Limit vehicle flow to the capacity of the bottleneck
- Reduce vehicle speed over the congested section

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*Personal communication, 1975.

- Extend the congestion upstream of the bottleneck
- Extend the duration of the period of peak emissions.

Intuitively, congestion should have the effect of greatly increasing the average vehicle emission rate per mile. Some freeway data supplied by Dr. May were analyzed to determine whether the average vehicle emission rate during congested flow (average speed of 20 mph) is much greater than the average emission rate at a similar speed from a vehicle driving the FTP driving cycle. The freeway data indicated ten full stops over a ten-mile length of roadway in which congestion occurred. Since the emission rate is largest when vehicles are in the acceleration mode (deceleration is similar to steady-state driving), it can be hypothesized that the greater the number of accelerations over a trip of given length, the greater the emission of pollutants over that trip. The FTP driving cycle shows 19 full stops in 7.5 miles. This would tend to indicate that driving in an uncoordinated street network is as bad or worse than driving over a congested section of a freeway. The intuitive assumption that the average vehicle will emit more pollutants when driving in congestion is correct simply because the average vehicle is traveling at a lower speed at a smaller spacing interval than normal and thus emits more pollutants per mile. An offsetting effect on vehicle emissions occurs because less vehicles (i.e., less than the roadway capacity) use the section of roadway during periods of congested flow. So although there is an increase in emissions on a macroscale (since excess demand is effectively being queued upstream), emissions may not be increased significantly on the microscale because of congestion. The effect of extending congestion upstream from the bottleneck has the effect of prolonging the period of congestion and causing congestion at offramps, onramps, and intersections that feed the freeway. Thus, a thorough analysis must account for all the upstream effects of congestion and the duration of these effects as volume demand increases to greater than capacity levels and then decreases to less than capacity.

The previous discussion was concerned with free-flow facilities such as freeways and expressways. The oversaturated or congested intersection also needs to be discussed. In the case of vehicles exiting from a sports stadium at the end of a game, it is possible to keep account of all vehicles waiting to be serviced at the stadium gates. The task becomes much more difficult, however, at an intersection that is part of a road network. Congestion at such an intersection may back up to an upstream intersection and cause congestion there. It will disrupt the progressive signal pattern of a group of coordinated signals; it will cause vehicles to choose alternative routes around the intersection; and it will affect cross traffic at the uncongested intersection approaches.

In general, the capacity of a signalized intersection is considered to be the maximum number of cars that can pass through it in a given period (usually one hour). An intersection differs from a freeway in that the intersection can operate at a volume to capacity ratio equal to 1.0 when demand volume exceeds capacity. Thus, there is no predictable reduction in the maximum number of vehicles that can proceed through the intersection when demand exceeds capacity. The vehicle spacing of queued vehicles limits the number of vehicles that are near a receptor at any time. As in the case of freeway congestion, emissions are spread over a longer period of time and longer distance, but the emission rate per mile is limited by the capacity of the roadway to store closely spaced vehicles. So again an apparent paradox can be stated: when vehicle demand is increased beyond capacity, the local effect may be no different than if vehicle demand equals capacity. On a macroscopic scale, the other intersections may become congested and vehicles will remain in the network for a longer period of time thus leading to higher 8-hour emissions and pollutant levels.

In summary, congested flow is beyond the scope of the Guidelines and more complex modeling techniques will have to be employed.